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WIND-INDUCED VIBRATION OF HIGH-RISE BUILDINGS ACCORDING TO DESIGN STANDARDS

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ABSTRACT

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Wind-induced vibration has become one of the main design challenges regarding modern high-rise buildings. High-rise buildings are naturally more sensitive for dynamic responses due to their low fundamental frequencies than low-rise buildings. At the same time, different wind conditions such as vortex effects intensify at higher levels and create more vibration prone structures.

The main problem regarding wind-induced vibration is occupants' respond to the fluctuation of a building and it has proven to cause various negative effects, such as sleepiness and motion sickness. For this reason, different criterion and methods to evaluate dynamic responses of structures are required.

The current problem regarding dynamic response evaluation of high-rise buildings is that most of the structural design standards offer limited information of this phenomenon, that due to its complexity, is hard to condense into a standardized form. This often leads to more advanced methods being used for estimating the building motion, such as wind tunnel tests, but different calculation estimates in the early designing process would potentially help to determine the need and scope for the vibration analysis.

Wind-induced vibration occurs in three directions, referred as the along-wind, cross-wind and torsional directions that are caused partially by differing circumstances. Extensive research shows that all direction and potentially the combinations of them induce notable risks for building fluctuation. Currently the dynamic response calculation method provided by the European Standard EN 1991-1-4 is relatively limited, and only offers a calculation method for the first direction of vibration response. However, in other standards around the world, new research of the subject has already been implemented. In this study, these different standards are evaluated and compared in terms of wind-induced vibration estimation and new methods to supplement the European Standard are searched.

Two comparison calculations are carried out in this study that describe the differences in the standards and their evaluation methods. Also, some challenges and potential problems in using standardized methods for these complex phenomena are discussed. Furthermore, relevant factors and building characteristic are evaluated, that create more vibration prone structures.

Keywords: wind loads, wind-induced vibration, high-rise building, building accelerations, wind code and standard

TIIVISTELMÄ

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Tuulivärähtely on muodostunut yhdeksi keskeiseksi suunnitteluhaasteeksi nykyaikaisten korkeiden rakennusten yhteydessä. Verrattuna mataliin rakenteisiin, korkeat rakennukset ovat luonnostaan herkempiä värähtelylle alhaisten ominaistaajuuksiensa vuoksi. Samanaikaisesti erilaiset tuuli-ilmiöt, kuten pyörrevirtaukset voimistuvat korkeammalle mentäessä ja saavat aikaan herkemmin värähteleviä rakenteita.

Merkittävä ongelma rakennusten tuulivärähtelyssä on asukkaiden suhtautuminen ilmiön vuoksi syntyvään huojumisliikkeeseen. Sen on todistettu aiheuttavat monia negatiivisia vaikutuksia ihmisten mukavuuteen, kuten esimerkiksi unettomuutta ja pahoinvointia. Tämän takia dynaamisten reaktioiden määrittämiseksi on oltava selkeitä kriteerejä, joilla rakenteita voidaan arvioida oleskelumukavuuden näkökulmasta.

Ongelma dynaamisten reaktioiden määrittämiselle korkeissa rakenteissa on aiheen suppea kattavuus nykyisissä suunnittelustandardeissa. Tämä johtuu osittain ilmiön moninaisuudesta, jota on vaikea tuoda tarpeeksi yksinkertaistetusti standarditasolle. Suppeat ohjeet johtavat usein tarpeeseen käyttää edistyneempiä arviointitapoja, kuten tuulitunnelikokeita, mutta erilaiset laskutavat suunnitteluprosessien alkuvaiheessa voisivat olla hyödyksi tuulivärähtelyn oleellisuuden ja mitoitustarpeiden arvioinnille.

Tuulivärähtelyä esiintyy kolmessa suunnassa, tuulen suuntaisena komponenttina, tuulen kanssa kohtisuorassa suunnassa ja tuulen vääntökomponenttina. Nämä ilmiöt aiheutuvat osittain eri syistä, ja kattava tutkimustieto aiheesta osoittaa, että jokainen komponentti ja mahdollisesti näiden yhdistelmät voivat aiheuttaa merkittävää värähtelyä rakenteissa. Nykyinen suunnitteluohje Euroopassa, EN 1991-1-4, kattaa aiheen melko suppeasti ja tarjoaa laskentatavan vain tuulen suuntaiselle värähtelykomponentille. Maailmanlaajuisesti aihetta on kuitenkin päivitetty osassa suunnittelustandardeja, minkä takia tässä työssä uusia suunnittelutapoja muiden maiden suunnittelustandardien mukaan on etsitty ja arvioitu, sekä etsitty täydentäviä laskutapoja, joita Euroopan normi ei vielä tarjoa.

Työssä on käytetty kahta esimerkkikohdetta osoittamaan eroavaisuuksia standardien laskutavoissa. Tämän lisäksi on esitetty tiettyjä haasteita aiheen standardisuunnittelussa ja tutkittu yleisellä tasolla rakenteiden ominaisuuksia, jotka alistavat niitä helpommin värähtelylle.

Avainsanat: tuulikuormat, tuulivärähtely, korkea rakennus, rakenteen kiihtyvyys, tuulistandardi

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PREFACE

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CONTENTS

1. INTRODUCTION ... 1
   1.1 Background and motivation ... 1
   1.2 Scope and structure of thesis ... 2
2. WIND-INDUCED VIBRATION IN BUILDINGS ... 3
   2.1 Wind-induced vibration ... 3
   2.2 Wind-induced torsional vibration ... 5
   2.3 Effects on low-rise and high-rise buildings ... 5
   2.4 The relevance of building cross-section and elasticity ... 6
   2.5 Current estimation methods and challenges in Finland ... 8
      2.5.1 Wind conditions in Finland ... 8
      2.5.2 SFS-EN 1991-1-4 ... 8
      2.5.3 Challenges and requirements for more advanced methods ... 13
3. STANDARD EVALUATION OF WIND-INDUCED VIBRATION ... 14
   3.1 Standard evaluation ... 14
   3.2 Limitations of the standards and requirements for more complex dynamic vibration analysis ... 14
   3.3 Dynamic behaviour in the along-wind direction ... 17
   3.4 Cross-wind accelerations ... 22
   3.5 Torsional accelerations ... 26
   3.6 Combining the dynamic components ... 29
   3.7 Previous studies comparing different standards ... 29
   3.8 Comparison between calculation parameters in standards ... 34
4. OTHER ESTIMATION METHODS FOR DYNAMIC RESPONSES OF TALL BUILDINGS ... 36
   4.1 NatHaz aerodynamic loads database ... 36
   4.2 Wind tunnel testing ... 41
5. HUMAN PERCEPTION OF WIND-INDUCED BUILDING MOTION ... 44
   5.1 ISO 10137 ... 44
   5.2 AIJ-GBV-2004 ... 46
6. COMPARISON AND CALCULATIONS ... 48
   6.1 CAARC Standard Tall Building ... 48
      6.1.1 Background and basic parameters ... 48
      6.1.2 Calculations of the along-wind accelerations ... 49
      6.1.3 Combined results and comparison of the CAARC building ... 52
   6.2 Tall building according to NALD Experimental Data ... 55
      6.2.1 Background and basic parameters ... 55
      6.2.2 Calculating the wind-induced vibration ... 56
      6.2.3 Combined results and comparison of the NALD example building ... 60
7. CONCLUSIONS ... 62

REFERENCES..64
APPENDIX A – Calculations of CAARC Standard Tall Building
APPENDIX B – Calculations of NALD Example Building

LIST OF FIGURES

Figure 1. *Components of wind-induced vibration 3*
Figure 2. *Building geometry and parameters according to SFS-EN 1991-1-4 ..... 10*
Figure 3. *Building geometry and parameters according to AIJ Recommendations 16*
Figure 4. *A 14-storey reinforced concrete building tested by Zdraveski and Mickoski [11] 30*
Figure 5. *Results of the 14-storey reinforced building in E-W direction of the wind for along-wind and cross-wind accelerations [11]......................... 31*
Figure 6. *Results of the 14-storey reinforced building in N-S direction of the wind for along-wind and cross-wind accelerations [11]......................... 32*
Figure 7. *Balsa Wood Models [12] 36*
Figure 8. *User interface of NatHaz Aerodynamic Loads Database [12]............... 37*
Figure 9. *User interface to add calculation values on NatHaz Aerodynamic Loads Database [12].. 38*
Figure 10. *Design wind speeds and RMS base bending moment coefficients according to NALD [12].. 39*
Figure 11. *Results for serviceability design according to NALD [12]...................... 40*
Figure 12. *Wind tunnel testing model [23]... 41*
Figure 13. *ISO 10137 standard on building serviceability...................................... 45*
Figure 14. *Combined peak acceleration criteria according to ISO 10137 and AIJ recommendations 46*
Figure 15. *Geometry of the CAARC building and two cases for calculation according to wind direction.. 48*
Figure 16. *Results calculated by SFS-EN 1991-1-4 for Case B using different slenderness factors, compared with the ISO 10137 and AIJ-GBV-2004 guidelines... 54*
Figure 17. *Geometry of the NALD example building 55*

LIST OF SYMBOLS AND ABBREVIATIONS

AIJ	The Architectural Institute of Japan
ASCE	American Society of Civil Engineering
BSL	Building standard laws of Japan
ISO	International Organization for Standardization
MDOF	Multi-degree-of-freedom
NALD	NatHaz Aerodynamic Loads Database
GLF	Gust Loading Factor

Symbols used in SFS-EN-1991-1-4:

b	width of the structure
h	height of the structure
ρ	air density
m_e	equivalent mass per unit length of the fundamental mode
$m_{1.x}$	along-wind fundamental equivalent mass
S_v	one-sided variance spectrum
S_L	non-dimensional power spectral density function
f_L	non-dimensional frequency
$n, n_{1.x}$	natural frequency of a structure
n_1	fundamental frequency
v_m	the mean velocity of the wind
L	turbulence length scale
I_v	turbulence intensity
z	measured height
z_t	reference height of 200 m
z_0	roughness length in meters
L_t	reference length scale of 300 m
α	non-dimensional factor
B	background factor
R	square root of resonance response factor
δ	total logarithmic decrement of damping
δ_s	total logarithmic decrement of structural damping
δ_a	total logarithmic decrement of aerodynamic damping for the fundamental node
δ_d	total logarithmic decrement of damping due special devices
c_f	force coefficient for wind action in wind direction
k_p	peak factor
K_x	non-dimensional coefficient
η	mode shape
v	up-crossing frequency
T	average time for mean wind velocity of 600 s
ζ	slenderness and shape factor of the structure
R_h , R_b	aerodynamic admittance functions
$\sigma_{a.x}$	standard deviation of characteristic along-wind acceleration
$a_{x.max}$	peak acceleration in the along-wind direction

Symbols used in AIJ-RLB-2015:

B	width of the structure
D	depth of the structure

H	mean roof height of the building
k	structural factor
$m(Z)$	mass per unit height
$i(Z)$	inertial moment per unit height
C_H	wind force coefficient at reference height
C'_g, C'_L, C'_T	rms overturning moment coefficient
f_D	natural frequency of the first mode in the along-wind direction
f_L	natural frequency of the first mode in the cross-wind direction
f_T	natural frequency of the first mode in torsional direction
α	exponent for power law for wind profile
β	exponent for power law for first translational vibration
$\beta_1, \beta_2, f_{s1}, f_{s2}$	structural factors
λ	mode correction factor of generalized wind force
ρ	air density
μ	first mode shape in each vibration direction
$\zeta_D, \zeta_L, \zeta_T$	damping factor for the first translational mode
$a_{D.max}$	peak acceleration in the along-wind direction
$a_{L.max}$	peak acceleration in the cross-wind direction
$a_{T.max}$	peak acceleration in the torsional direction
g_{aD}	peak factor for along-wind vibration
g_L	peak factor for cross-wind vibration
g_T	peak factor for torsional vibration
E_{gl}	topography factor
I_H	turbulence intensity at the reference height
I_T	generalized mass of the building for torsional vibration
I_{rH}	turbulence intensity at reference height
L_H	turbulence scale at reference height
U_H	design wind speed
F	wind force spectrum factor
F_D	along-wind spectral factor
F_L	cross-wind spectral coefficient of overturning moment
F_T	torsional spectral coefficient
R	correlation coefficient between wind pressure on the windward and leeward faces
R_D, R_L, R_T	resonance factor
S_D	size effect factor
M_D	generalized mass of the building for along-wind
M_L	generalized mass of the building for cross-wind
M_T	generalized mass of the building for torsional vibration
q_H	design velocity pressure
Z_b	parameter of exposure factor

Symbols used in AS/NZS 1170.2-2002:

s	reference height
k	mode shape power exponent for the fundamental mode
B	breadth of the structure
H	mean roof height of the building
S	size reduction factor
N	reduced frequency
Δz	height of the section upon which the wind pressure acts
ξ	ratio of structural damping to critical damping of structure

$v(z)$	orthogonal design wind speed at the height z
$v(H)$	orthogonal design wind speed at the height H
v_{des}	design wind speed
V_n	reduced velocity
b_{sH}	average breadth of the structure between s and H
b_{0H}	average breadth of the structure between 0 and H
B_z	average breadth of section at height z
C_{fs}	crosswind force spectrum coefficient
C_l	aerodynamic shape factor in leeward direction
C_w	aerodynamic shape factor in windward direction
C_{dyn}	dynamic response factor
I_H	turbulence intensity
E_t	spectrum of turbulence
g_v	peak factor for the upwind velocity fluctuations
K_m	mode shape correction factor for crosswind acceleration
B_s	background factor
H_s	height factor for the resonant response
g_R	peak factor for the resonant response
L_H	measure of the integral turbulence length scale at height H
n_a	first mode natural frequency of vibration in the along-wind direction
n_c	first mode natural frequency of vibration in the cross-wind direction
v_{des}	building orthogonal design wind speed determined at the height H
$a_{x.max}$	peak along-wind acceleration at the top of the building
$a_{y.max}$	peak cross-wind acceleration at the top of the building
M_0	average mass per unit height
ρ_{air}	density of air

.

1. INTRODUCTION

1.1 Background and motivation

Wind-induced vibration has become an important phenomenon considering high-rise building designing. Modern buildings tend to be taller than in previous decades, due to trends as urbanization and lack of land space in cities. Also, global warming has increased some extreme weather events that will serve new challenges for the structural engineering field in the following years.

High-rise buildings are more sensitive for dynamic responses due to their lower natural frequencies compared to low-rise buildings. Simultaneously, when the height of the building increases, different wind conditions such as vortex effects intensify and create more vibration prone structures. Wind-induced vibration in high-rise buildings has caused structural failures, but the main problem that arises is the occupants' comfort. Humans respond to the fluctuation of a building and it has proven to cause many negative effects, such as difficulties to perform everyday tasks, cause sleepiness, reduce work performance and induce motion sickness. For this reason, different criterion and methods to evaluate dynamic responses of structures are required.

Wind-induced vibration is divided into three components that are referred as the along-wind, cross-wind and torsional components. Most evaluation methods have focused on the first component of vibration, but studies have shown, that especially high-rise buildings respond significantly also to the cross-wind and torsional directions. Therefore, when designing a modern high-rise building, all components of vibration might require careful evaluation.

Currently the amount of information about wind-induced vibration given by the European Standard EN 1991-1-4 is relatively small considering the complexity of this phenomenon. The standard only offers a procedure to calculate the along-wind accelerations, which is fairly typical problem in many structural designing standards and codes around the world. In some countries however, new research around wind-induced vibration has already been implemented to the designing standards that offer alternative solutions for the current designing problems.

The main purpose of this study is to compare methods and scopes to evaluate wind-induced vibration for high-rise buildings according to different designing standards around the world. The current criteria for building motion and its effects are also evaluated. All three components of the vibration are considered, and special focus is given to the least studied component known as the torsional vibration.

1.2 Scope and structure of thesis

This study focuses only on the wind-induced vibration of high-rise buildings and does not cover different causes of vibrations, such as seismic activity. Wind-induced vibration is first investigated by going over current literature and research regarding the phenomenon. Then some of the main designing standards around the world are evaluated and their methods to calculate wind-induced vibration is compared. Lastly, two calculation examples are carried out using the SFS-EN 1991-1-4 standard and AIJ Recommendations for Loads on Buildings, that are then compared with previous results and comfort guidelines.

The chapter 2 covers the theoretical background for the phenomenon and its effects on structures. Some of the main building characteristics are discussed, that are especially sensitive for dynamic responses. Also, the current designing methods according to the European Standard used in Finland is given as a comparison background for the further chapters.

In the chapter 3 different structural designing standards are evaluated. Attention is given especially for standards that cover more than the along-wind direction of vibration. The scopes of standards and their designing criteria are evaluated as well. Some advanced methods to evaluate high-rise buildings outside designing standards are offered in the chapter 4.

In the chapter 5 current criterion and guidelines for vibration are evaluated and the human perception of wind-induced building motion is discussed.

In the chapter 6 two example calculations are carried out using the calculation methods of designing standards covered in the previous chapters. Calculations are meant to describe some of the differences further and give more context for the study. Important differences in the standards are compared further and the importance of selecting parameters from the standards is highlighted.

2. WIND-INDUCED VIBRATION IN BUILDINGS

When estimating wind loads for a structure in standard based design, the loads are usually considered as perpendicular, fixed forces distributed evenly on the surface of the building. Although this simplified approach is often enough to determine the impact of the wind on a necessary level, the wind/structure interaction is a far more complicated phenomenon than that due to winds fluctuating nature. Evolution of designing concept has made the modern high-rise buildings typically more wind sensitive and therefore more advanced methods to estimate these aerodynamic effects are often required.

2.1 Wind-induced vibration

The fluctuating nature of the wind causes resonant dynamic responses in structures of which natural frequencies and damping are low. This vibratory response can be divided into three main components: along-wind component or drag, cross-wind component or lift and torsional component as shown in Fig. 1. There are many factors that affect the experiences of resonant dynamic response and their distribution, such as the shape, mass and stiffness of the structure. [1, s. 113-114, 210]

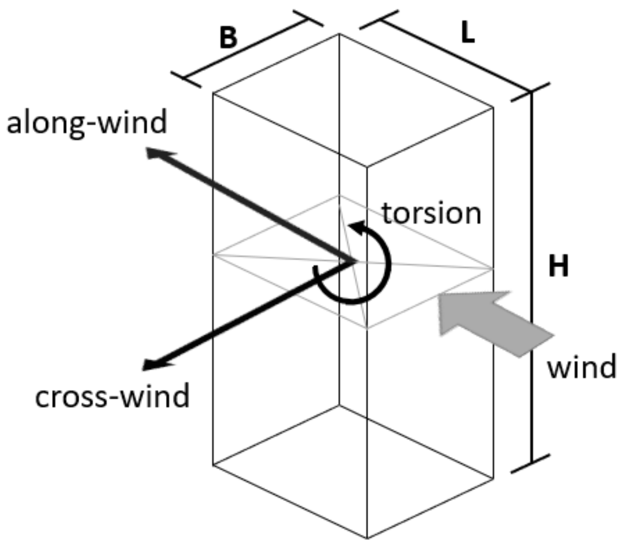

Figure 1. *Components of wind-induced vibration*

Generally, the along-wind forces are the primal cause of deflection, moments and shears in the structural frame, which results in design procedures and standards focusing on this oscillation way. The natural turbulent velocity fluctuation of the wind is the main cause of dynamic response in the along-wind direction. [1, s. 230]

In the cross-wind direction the response is more complicated to predict, since it is created by random vortex shedding. It means that the building is generating an unsteady separating flow by itself with contribution from cross-wind turbulence. [1, s. 230]

The torsional component of the dynamic response is the least known of these three components but its significance in tall buildings has been highlighted since a study of the 293 meter tall Commerce Court building in Canada in the 1970s, which is one of the most detailed and well-documented studies of a tall building to date. The study included wind-tunnel testing and a multi-degree-of-freedom (MDOF) aeroelastic model and tests were executed in both the designing stage and later for a pressure model. One feature noticed in the test was a significant torsional motion in one direction (east-west), which was explained by eccentricity between the mass and elastic axis in the north-south direction of the building. [1, s. 218-219]

Considering the components different characteristics and source, it is safe to say that the three components of wind-induced vibration can occur simultaneously. The question when dealing with the dynamic responses is therefore: how can we combine these different components statistically? [1, s. 232-234]

In structures, such as high-rise buildings, where all three dynamic components of the wind are a relevant cause of building accelerations, it is common to evaluate the two orthogonal lateral forces and the torsional moment independently. If the mass and elastic centres of the structures are coincident and the three wind components are uncorrelated, this analysis is valid. In the case of asymmetric structure however, the centres of aerodynamic force, mass and stiffness are noncoincident. Thus, the cross-wind and torsional components of the wind can cause instabilities and are statistically correlated. [5]

This results that in some cases, treating these three factors separately as different load cases is outdated and even potentially dangerous design solution. [1, s. 232-234] Current building codes and standards do not provide a clear answer of how to combine these components and many of them do not provide a method to calculate the cross-wind or torsional components at all. However, there are some methods to combine the along-wind and cross-wind components, that have been proven to produce quite accurate results. The most accurate predictions are still by carrying out a specific wind-tunnel test, which allow exact measures for the building shape, surroundings and mode shapes. More detailed analysis of wind-tunnel testing is provided in the chapter 4.

2.2 Wind-induced torsional vibration

In this research all components of the dynamic responses are studied, but the main focus is on the third, least studied component described as the torsional vibration. As noticed in the studies of the Commerce Court building, buildings that have eccentricity between the elastic and mass centres can generate significant dynamic twist, that will increase its accelerations. This effect has been overlooked in studies around dynamic responses of the wind loads, which have been focused more on the along-wind and cross-wind directions. [1, s. 233]. Later studies have shown that also turbulence buffeting, vortex shedding and re-attaching flow onto a long afterbody can result in dynamic torsional vibrations [2, s.141].

Holmes [1] divided the cause of dynamic torque and torsional motions into two different mechanisms:

1. torsional motion resulting from non-uniform pressure distributions, including non-symmetric cross-sectional geometries
2. torsional motion resulting from sway motions through coupled shapes and/or eccentricities between shear and geometric centres. [1]

Estimating torsional loads and motions is still largely analytical and for that reason it has not reached adequately advanced stage to be included in most current design codes and standards. However, its importance in the design process has increased significantly in consequence of more complex shapes and structural systems in modern buildings. [2, s. 141] There are different approximation techniques to identify a general idea of the dynamic response in the torsional direction of a building, and to identify the need for more detailed wind-tunnel procedure or MDOF aeroelastic model, which are discussed in more detail in the following chapters.

2.3 Effects on low-rise and high-rise buildings

Emporis Standards [4] defines a low-rise building as an enclosed structure below 35 meters in height or maximum of 11 floors. A building of unknown height from 12-39 floors or between 35 and 100 meters in height is a high-rise building. Above 100-meter or 39 floors structures are defined as skyscrapers. [4]

As a comparison, Holmes [1] defines a low-rise building as a roofed structure which is less than 15 meters in height. He states that for these types of buildings, resonant dynamic effects can usually be disregarded in the estimation process. [1, s. 193] While defining a need for dynamic response evaluation however, the building material should also be considered, as it has a great impact on the intensity of vibration. For example, timber structures have lower mass and stiffness values compared to similar structures made from steel and concrete, which makes high-rise timber buildings more prone for vibration. [28]

In Finland, over 12 floor multi-story structure is usually classified as a high-rise building. Therefore, determining the need for more detailed dynamic response estimation cannot be done only based on the floor number of the structure but also considering the overall height and moreover the shape, material and floor layouts of the building.

Since cross-wind and torsional vibrations are both caused by the different vortex effects on the building, the vibrations on these directions are typically not significant in a low-rise building. However, when the building height increases, the vortex effects intensify as well and increase the cross-wind and torsional response. Simultaneously, the natural frequency of the building decreases with the height, which leads to more vibration prone structures. For this reason, the cross-wind and torsional responses start to increase faster than the along-wind responses, when the building is a target for higher wind speeds. [16, s. 246] Under normal wind conditions the along-wind responses are usually larger for buildings than in the cross-wind direction. However, under storms and other extreme circumstances, it is not rare that the cross-wind responses get significantly larger than in the along-wind direction. This is demonstrated later in the chapter 4 and 6, where different examples are studies and calculations of the peak accelerations are estimated.

2.4 The relevance of building cross-section and elasticity

The symmetricity of a structure can be described in two ways: by the centre of mass and by the centre of rigidity, also known as the elastic centre. It is important to understand the difference between these two, especially when considering the torsional responses of a structure, since the distance between the mass and rigid centres is one of the key factors to increase torsional effects. The distance between a mass centre and a centre of rigidity is referred as the eccentricity of the building.

Zhang et al. stated in their research in 1993 [6] that a small increase in buildings eccentricity can cause a notable rise in the mean twist angle and dynamic torsional response. A building with square cross-section is sufficient for double the mean angle of twist and 40-50% increase of dynamic twist, when the elastic centre is moved 10% from the mass centre. [6] In a regular high-rise building the eccentricity is found to be from 10% to 15% and for some torsionally sensitive buildings it can rise to more than 20% [7].

The floor plan geometry of the structure is one of the most important factors to effect on the building's torsional behavior. Repeated floor plan throughout the whole building is easier when predicting the torsional responses, but often modern structures have asymmetric shapes and diverse floor layouts. Also, the building stiffness system and the surrounding structures might increase or decrease the torsional loads of the building. One torsion-prone building shape is an L-shape building, that has high potential for large eccentricity between the two centers. [7]

The shape of the building is overall a large factor in all three dynamic response directions. A study to estimate the optimal building shape was conducted in Pittsburgh, U.S. in the 1971 [1], where six buildings of identical height but different cross-sections were investigated in a boundary-layer wind tunnel. The different shapes estimated were circular, 2:1 rectangular in both axis towards the wind direction, square, square with chamfered corners and triangular cross-section. The lowest response was produced for the circular cross-section, followed by the rectangular cross-section with the stronger axis towards the wind. The triangular cross-section produced the highest motion response. Also, the deflection on the shorter axis of a 2:1 rectangular cross-section was significantly larger compared to the same shape with stronger axis facing the wind direction. [1, s. 229]

The study also showed a difference in the two square cross-sections. The square cross section with chamfered corners produced lower response than the one without corner modifications. Kwok et al. [24] investigated that chamfered corners of 10% of the building width would reduce the along-wind responses by up to 40% and cross-wind responses by 30%. [24]

Sometimes the width and length of the building change in the vertical direction, meaning that the shape in the higher parts of the structure is different than in the lower parts. In this case the wind load impact on the upper parts affects more on the vibration response of the building. Therefore, while using standardize equations in calculations, special attention should be applied to the elasticity of the structure in the upper parts of the building. [16, s. 246]

2.5 Current estimation methods and challenges in Finland

High-rise buildings have still been rare in Finland compared to many other countries, but the recent years have shown that modern buildings tend to become higher. Due to urbanization and lack of free land space in bigger cities, this trend can be assumed to continue.

2.5.1 Wind conditions in Finland

The definition of a storm in Finland requires the 10-minute mean wind velocity to reach at least 21 m/s on one sea station. On average there are 15 days in a year when the wind on the sea reaches this mean velocity. [14]

The highest 10-minute mean wind velocity in Finland was measured in 2019 in Bogskär. The velocity was 32,5 m/s and it was measured on a sea station. Therefore, the 10-minute mean wind velocity in Finland has never reached the level of a hurricane (33 m/s). The peak wind velocities in Finland have been measured on top of mountains in Lapland and they have reached the velocity of 50 m/s. [14]

The structural design standard that addresses the wind actions in Finland is the SFS-EN 1991-1-4, which is a part of the European Standard Eurocode 1. All countries using the European Standard complement it with the national annexes (NA) concerning their country. The Finnish NA determines the fundamental value of basic 10-minute wind velocity to be 21 m/s [15].

As a comparison, in Japan the minimum basic 10-minute mean wind speed is set to 30 m/s and it varies up to 50 m/s depending on the location [16]. In America, the basic wind speed is determined by 3-second gust wind speeds at 10 meter above the ground and varies from 38 m/s to 76 m/s [17].

When comparing different standards and designing codes, understanding the circumstances used in designing is important. If one standard uses 10-minute mean wind speed as the designing basis and another uses 3-second gust speed, this difference should be noted when comparing the results.

2.5.2 SFS-EN 1991-1-4

Calculating wind-induced vibration is still new in Finland and estimation rely heavily on calculation models and sometimes wind tunnel tests. The current challenge is that SFS-EN 1991-1-4 only provides a way to calculate the displacement and accelerations in the along-wind direction. Two methods for this are offered in the Annex B and Annex C.

European standard also describes dynamic characteristics of structures in the Annex F and gives some equations to calculate the fundamental dynamic properties, such as natural frequencies and logarithmic decrements of damping. Its procedures assume that the designed structure possess linear elastic behaviour and classical normal modes. [10]

The Annex B of SFS-EN 1991-1-4 starts the calculation procedure for the along-wind accelerations by determining the effects of wind turbulence. This is done by calculating the turbulence length $L(z)$, which represents the natural fluctuation of the wind. The turbulence length is then used to calculate the wind distribution over frequencies, which is expressed by a non-dimensional power spectral density function $S_L(z,n)$:

$$S_L(z,n) = \frac{n \cdot S_v(z,n)}{\sigma_v^2} = \frac{6{,}8 \cdot f_L(z,n)}{\left(1 + 10{,}2 \cdot f_L(z,n)\right)^{5/3}} \tag{2.1}$$

where $S_L(z,n)$ is the one-sided variance spectrum
$f_L(z,n)$ is a non-dimensional frequency calculated as:

$$f_L(z,n) = \frac{n \cdot L(z)}{v_m(z)} \tag{2.2}$$

where n is the natural frequency of the structure in Hz
$v_m(z)$ is the mean velocity
$L(z)$ is the turbulence length scale on height z:

$$L(z) = L_t \cdot \left(\frac{z}{z_t}\right)^{\alpha} \quad \text{for} \quad z \geq z_{min} \tag{2.3}$$

$$L(z) = L(z_{min}) \quad \text{for} \quad z < z_{min}$$

where z_t is a reference height $z_t = 200\ m$
L_t is a reference length scale of $L_t = 300\ m$
α is a factor where $\alpha = 0{,}67 + 0{,}05 \ln(z_0)$
z_0 is the roughness length in meters

After that, the structural factor is defined. The background factor B^2 expresses the lack of full correlation of the pressure on the surface of the structure. It considers the building size and the effects of wind turbulence on the surface. Setting the background factor as 1 gives safe results.

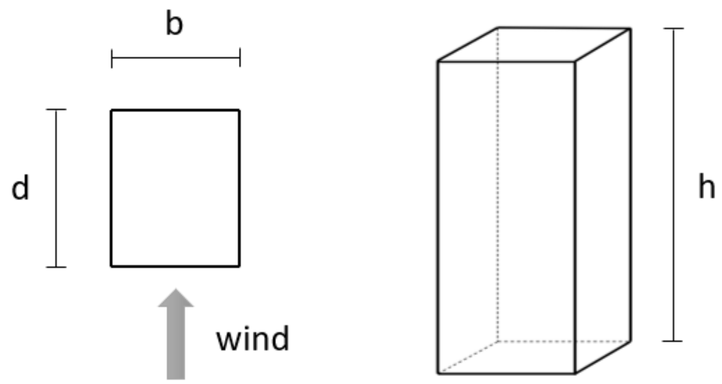

Figure 2. *Building geometry and parameters according to SFS-EN 1991-1-4*

$$B^2 = \frac{1}{1 + 0{,}9 \cdot \left(\frac{b + h}{L(z_s)}\right)^{0{,}63}} \tag{2.4}$$

where	b	is the width of the building
	h	is the height of the building
	$L(z_s)$	is the turbulence length scale on height (2.3)

The resonance response factor R describes the turbulence in resonance with the structures vibration mode and it is calculated using the following expressions:

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot R_h(n_h) \cdot R_b(n_b) \tag{2.5}$$

where	δ	is the total logarithmic decrement of damping (2.6)
	S_L	is the non-dimensional power spectral density function
	R_h, R_b	are the aerodynamic admittance functions (2.8)

$$\delta = \delta_s + \delta_a + \delta_d \tag{2.6}$$

where	δ_s	is the total logarithmic decrement of structural damping
	δ_a	is the total logarithmic decrement of aerodynamic damping for the fundamental node
	δ_d	is the total logarithmic decrement of damping due special devices (TMD etc.)

As seen above, European Standard treats the structural damping on a logarithmic scale, which is a clear difference from some other designing standards around the world. According to Eurocode, in most cases δ_a for alongwind vibrations can be estimated by simplified equation (2.7) that presumes that the modal deflections of the structure are constant for each height z. If this is not the case, the equivalent mass per unit area of the structure will replace the equivalent mass per unit length in the equation, which will be a more accurate representation of the mass distribution of the structure.

$$\delta_a = \frac{c_f \cdot \rho \cdot b \cdot v_m(z_s)}{2 \cdot n_1 \cdot m_e} \tag{2.7}$$

where	c_f	is the force coefficient for wind action in wind direction
	n_1	is the fundamental frequency, which for high-rise buildings can be estimated as $n_1 = 46/H$
	m_e	is the equivalent mass per unit length of the fundamental mode

The aerodynamic admittance functions required in the expression (2.5) can by determined by:

$$R_h = \frac{1}{\eta_h} - \frac{1}{2 \cdot {\eta_h}^2}(1 - e^{-2 \cdot \eta_h}); \qquad R_h = 1 \quad \text{for} \quad \eta_h = 0 \tag{2.8}$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot {\eta_b}^2}(1 - e^{-2 \cdot \eta_b}); \qquad R_b = 1 \quad \text{for} \quad \eta_b = 0$$

where the mode shapes η are determined by expressions (2.9). However, in case of mode shapes with internal node points Eurocode advices more detailed calculations to be used.

$$\eta_h = \frac{4{,}6 \cdot h}{L(z_s)} \cdot f_L(z_s, n_{1,x}) \tag{2.9}$$

$$\eta_b = \frac{4{,}6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x})$$

Now using the background factor and the resonance response factor, the up-crossing frequency v of the building can be determined. This is then used to calculate the peak factor k_p which is defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation. This is used later in the equation (2.15) to define the peak acceleration for the structure.

$$v = n_{1.x}\sqrt{\frac{R^2}{B^2 + R^2}} \quad ; v \geq 0{,}08\ Hz \tag{2.10}$$

where $n_{1.x}$ is the natural frequency of the structure

$$k_p = \sqrt{2 \cdot \ln(vT)} + \frac{0.6}{\sqrt{2 \cdot \ln(vT)}} \tag{2.11}$$

where T is the average time for the mean wind velocity = 600 seconds

The fundamental flexural mode in along wind direction for buildings is:

$$\Phi_1(z) = \left(\frac{z}{h}\right)^{\zeta} \tag{2.12}$$

where ζ is a factor determined by slenderness and shape of the structure, varying from 0,6 to 2,5

If the expression 2.10 is used to determine the fundamental mode, can the non-dimensional coefficient K_x next be determined as:

$$K_x = \frac{(2 \cdot \zeta + 1) \cdot \left\{(\zeta + 1) \cdot \left[\ln\left(\frac{z_s}{z_0}\right) + 0{,}5\right] - 1\right\}}{(\zeta + 1)^2 \cdot \ln\left(\frac{z_s}{z_0}\right)} \tag{2.13}$$

Now the standard deviation of the characteristic along-wind acceleration of the structure at the height z can be obtained by the following expression:

$$\sigma_{a,x}(z) = \frac{c_f \cdot \rho \cdot b \cdot l_v(z_s) \cdot {v_m}^2(z_s)}{m_{1,x}} \cdot R \cdot K_x \Phi_{1,x}(z) \tag{2.14}$$

where c_f is the force coefficient

ρ	is the air density
$l_v(z_s)$	is the turbulence intensity
$v_m(z_s)$	is the mean wind velocity
z_s	is the reference height
R	is the square root of resonant response
K_x	is the non-dimensional coefficient
$m_{1,x}$	is the along wind fundamental equivalent mass

The peak acceleration in the along-wind direction can now be calculated by multiplying the standard deviation of the acceleration with the peak factor given in (, using the natural frequency as the up-crossing frequency as $v = n_{1,x}$.

$$a_{x.max}(z) = \sigma_{a,x}(z) \cdot k_p \tag{2.15}$$

2.5.3 Challenges and requirements for more advanced methods

While Eurocode provides a detailed method to calculate the accelerations in the along-wind directions, it still requires many simplification and assumptions. If the structure does not possess linear elastic behaviour, for example if the floor plan varies in the structure or the shape is otherwise asymmetrical, the equations cannot be used. Eurocode also does not mention the torsional vibration and does not consider how the three accelerations components might behave in combination. It also does not provide guidance when more than the fundamental mode needs to be considered in vibration or if the structure is higher than 200 meters. [10]

Therefore, it would be important to utilize more advanced calculation methods for high-rise buildings that would characterize the dynamic behaviours better. Simplified methods to evaluate possible need for more throughout investigation would be helpful for structural engineers, especially early in the designing process.

In the next chapters different buildings codes around the world are valued and compared to the estimation methods provided by Eurocode. Also, some analytical methods outside standards evaluation are discussed, as well as further challenges in understanding the dynamic behaviours.

3. STANDARD EVALUATION OF WIND-INDUCED VIBRATION

3.1 Standard evaluation

Due to the intensive research around wind designing of buildings in the last century, there has generally been significant development in multiple wind standards and codes around the world. However, there are always new problems to be settled and new methods to be implemented. The effects caused by wind for tall and slender structures can be described as one of the current problems, as well as the evaluation of wind loads on buildings of different shapes, evaluation of internal pressures and the effects of upstream terrain roughness. [2]

Standard evaluation focuses mostly on low-rise buildings and different numerical evaluation methods are still required when the designed structural system is more complex [2]. As concluded in the chapter 2, a high-rise building is usually a more challenging structure that requires time-domain numerical analysis and digitally simulated data. Wind-induced torsion and wind-induced vibration in general becomes a significant phenomenon usually only for taller and slender buildings, which is why there are not many mentions of it in different standards around the world. In this chapter different building codes and standards are studied and mentions of wind-induced vibrations are collected.

One factor to note when comparing different structural designing standards around the world, is to also pay attention to the minimum values of basic wind speeds. Strong winds, tropical cyclones or tornados might be regular phenomena in some countries, whereas they are uncommon in more Northern countries like Finland. For this reason, understanding the large-scale wind phenomena of a country is important when comparing different standards with each other.

3.2 Limitations of the standards and requirements for more complex dynamic vibration analysis

Standards describe the requirements for more in depth analysis of dynamic responses in different ways. Some, like Eurocode, do not give clear indications of when to implement more advanced methods, such as determining the torsional vibration. It does however state clearly that when more than the fundamental mode of the vibration needs to be considered, its methods cannot be implemented. Also structures that do not behave linearly do not fill its requirements. [10]

The building standard used in America is called ASCE (American Society of Civil Engineering). It describes a flexible structure as one of which fundamental natural frequency is less than 1 Hz. ASCE also requires a structural model and analysis for this type of building that accounts for mass distribution, stiffness and damping. [17]

Similarly, the Australian/New Zealand Standard (AS/NZS) defines that tall buildings and towers that's natural first mode fundamental frequencies are less than 1 Hz require along-wind and crosswind response analysis. It also limits the buildings with frequency less 0.2 Hz to not be covered in the standard. [20]

AS/NZS also provides a limit for motion serviceability for wind sensitive buildings, that indicated that the acceptable crosswind acceleration levels may be exceeded if the following conditions are true:

$$\frac{H^{1.3}}{M_0} > 0.0016 \tag{3.1}$$

where H is the average roof height of a structure above the ground
M_0 is the average mass per unit height in kg/m [20]

The Architectural Institute of Japan (AIJ) has published many design codes, manuals and recommendations. Many of them are approved by the Japanese government but some are still not completely consistent with the Building Standard Law of Japan (BSLJ) and consequently their use for load evaluation is generally restricted. BSLJ describes the minimum building design requirements in Japan and has been sifting to Performance Based Design in the latest revision. Design approval is carried out by a selected organization on behalf of the Minister of Land, Infrastructure and Transport (MLIT). [8,9]

BSLJ-2000 is the most recent revision of the Japanese building standard and it identifies the requirements for a design procedure in different occasions. Unlike Eurocode and other most of the other building standards, it specifies along-wind, cross-wind and torsional loads individually. [9]

BSLJ specifies that buildings over 60 meters in height or slender buildings that fill the conditions of the following formula require analysis of the across-wind and torsional loads [9].

$$\frac{H}{\sqrt{BD}} \geq 3 \tag{3.2}$$

where H is building height
B is building width
D is building depth

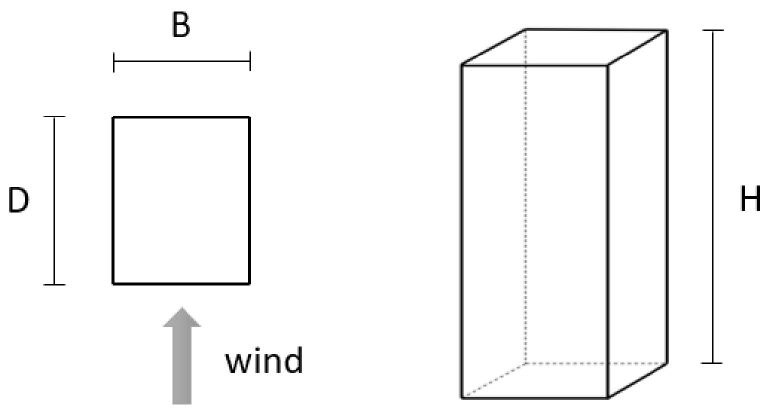

Figure 3. *Building geometry and parameters according to AIJ Recommendations*

The problem is that BSL describes the requirement for crosswind and torsional analyse for high-rise building over 60 meters in height but does not present any methods for calculation. AIJ recommendations do however, which leads structural designers in Japan commonly using them for more advanced buildings. [9]

The latest version of AIJ recommendations is from the year 2015 and its estimation methods for the cross-wind and torsional directions can be used for structures that fill the following conditions:

1. The cross section is rectangular without change in the vertical direction
2. $\frac{H}{\sqrt{BD}} \leq 6$
3. $0.2 \leq \frac{D}{B} \leq 5$
4. $\frac{U_H}{f\sqrt{BD}} \leq 10$

Where D is the building depth (in wind direction), U_H is the design wind speed and f is the natural frequency of the first mode of the structure in either cross-wind or torsional directions. [16]

Most structural designing codes use serviceability design for the acceleration evaluation. The averaging time for vibration and return periods for wind velocities vary, and this is discussed further in the following chapters.

3.3 Dynamic behaviour in the along-wind direction

Estimating the dynamic response in the along-wind direction has been covered and explained thoroughly in various building standards around the world. The calculation methods given in Eurocode have already been covered in the chapter 2.5. As a comparison, next the along-wind response calculation method is presented according to the latest AIJ Recommendations and the 2002 version of AS/NZS. After going over the equations, some studies are presented, which have compared different standardized methods with monitored data and wind tunnel tests.

AIJ-RLB-2015

According to AIJ-RLB-2015 the maximum response acceleration (m/s^2) at the top of the building in the along-wind direction is calculated using the following equation:

$$a_{Dmax} = \frac{q_H g_{aD} B H C_H C'_g \lambda \sqrt{R_D}}{M_D} \tag{3.3}$$

where

g_{aD}	is the peak factor for along-wind vibration as defined in (3.4)
q_H	is the design velocity pressure as $\frac{1}{2}\rho U_H^2$ where ρ is the air density (assumed to be 1.22) and U_H the design wind speed
B	is the building width
H	is the mean roof height of the building
C_H	is the wind force coefficient C_D at reference height
C'_g	is the rms overturning moment coefficient as defined in (3.5)
λ	is the mode correction factor of generalized wind force as defined in (3.8)
R_D	is the resonance factor as defined in (3.9)
M_D	is the generalized mass of the building for along-wind as defined in (3.14)

Now the required expressions are listed in the following formulas. First, the peak acceleration in the along-wind direction can be calculated as:

$$g_{aD} = \sqrt{2\ln(Tf_D) + 1.2} \tag{3.4}$$

where f_D is the natural frequency of the first mode in along-wind direction

T is the time of evaluation (600 s according to AIJ)

AIJ explains that the natural frequency f_D of the building is a critical factor in terms of accelerations, and it is usually estimated using an appropriate wind tunnel test or calculation model. Now the rms overturning moment coefficient is calculated as:

$$C'_g = 2I_H \frac{0.49 - 0.14\alpha}{\left\{1 + \frac{0.63\left(\sqrt{BH}/L_H\right)^{0.56}}{(H/B)^k}\right\}} \tag{3.5}$$

where I_H is the turbulence intensity at reference height given by (3.6)

α is the exponent of power law for wind profile as defined in a Table in standard (value from 0.1 to 0.35 depending of the terrain category)

k is a structural factor, $k = 0.07$ if $H/B \geq 1$ and $k = 0.15$ if $H/B < 1$

L_H is the turbulence scale at reference height given by (3.7)

$$I_H = I_{rH}E_{gl} \tag{3.6}$$

where I_{rZ} is the turbulence intensity at reference height for each terrain category defined in the standard

E_{gl} is the topography factor defined in the standard

$$L_H = \begin{cases} 100\left(\frac{H}{30}\right)^{0.5} \\ 100\left(\frac{Z_b}{30}\right)^{0.5} \end{cases} \tag{3.7}$$

where Z_b is a parameter of exposure factor defined in the standard (value ranges from 3 to 30 depending of the terrain category)

Next the mode correction factor can be defined as:

$$\lambda = 1 - 0.4 \ln \beta \tag{3.8}$$

where β is the exponent of power law for the first translational vibration mode in along-wind direction

Now the resonance factor for along-wind direction R_D is calculated. Also the along-wind spectral factor F_D and size effect factor S_D are required in the equation:

$$R_D = \frac{\pi F_D}{4\zeta_D} \tag{3.9}$$

where F_D is the along-wind spectral factor given by (3.10)
ζ_D is the damping factor for the first translational mode in along-wind direction

$$F_D = \frac{I_H^2 F S_D \left(0.57 - 0.35\alpha + 2R\sqrt{0.053 - 0.042\alpha}\right)}{{C'_g}^2} \tag{3.10}$$

where R is the correlation coefficient between wind pressure on the windward and leeward faces, defined by (3.11)
F is the wind force spectrum factor defined by (3.12)
S_D is the size effect factor defined by (3.13)

$$R = \frac{1}{1 + 20\frac{f_D B}{U_H}} \tag{3.11}$$

$$F = \frac{4\frac{f_D L_H}{U_H}}{\left\{1 + 71\left(\frac{f_D L_H}{U_H}\right)^2\right\}^{5/6}} \tag{3.12}$$

$$S_D = \frac{0.9}{\left\{1 + 6\left(\frac{f_D H}{U_H}\right)^2\right\}^{0.5} \left(1 + 3\frac{f_D B}{U_H}\right)} \tag{3.13}$$

where U_H is the design wind speed
f_D is the natural frequency of the first mode in along-wind direction
L_H is the turbulence scale at reference height given by (3.7)

Finally, the only undefined expression from the peak acceleration equation is the generalized mass of the building M_D which can be calculated with the following equation:

$$M_D = \int_0^H m(Z)\mu^2(Z)dZ \tag{3.14}$$

where $m(Z)$ is the mass per unit height (kg/m)
μ is the first mode shape of building in each direction

AS/NZS 1170:2-2002

The Australian/New Zealand Standards starts the estimation from the dynamic response factor C_{dyn} which is calculated as follows:

$$C_{dyn} = \frac{1 + 2I_H\left[g_v^2 B_s + \frac{H_s g_R^2 S E_t}{\xi}\right]^{0.5}}{(1 + 2g_v I_H)} \tag{3.15}$$

where s stands for the level at which action effects are calculated for a structure
H stands for the average roof height of the structure
g_v is the peak factor for the upwind velocity fluctuations, which is usually taken as 3.7
B_s is the background factor, caused by low frequency wind speed variation, given by (3.16)

H_s	is the height factor for the resonant response which equals as $1 + \left(\frac{s}{H}\right)^2$
g_R	is the peak factor for resonant response which equals as $\sqrt{[2 \times \log_e(600 n_c)]}$
S	is the size reduction factor given by (3.17)
E_t	is the spectrum of turbulence given by (3.18)

$$B_s = \frac{1}{1 + \frac{\left[0.26(H-s)^2 + 0.46 b_{sH}^2\right]^{0.5}}{L_H}} \tag{3.16}$$

where

b_{sH}	is the average breadth of the structure between s and H
L_H	is a measure of the integral turbulence length scale at height H which equals to $85\left(\frac{H}{10}\right)^{0.25}$

$$S = \frac{1}{\left[1 + \frac{3.5 n_a H\left(1 + g_v I_H\right)}{v_{des}}\right]\left[1 + \frac{4 n_a b_{0H}\left(1 + g_v I_H\right)}{v_{des}}\right]} \tag{3.17}$$

where

n_a	is the first mode natural frequency of vibration of a structure in the along-wind direction in Hertz
b_{0H}	is the average breadth of the structure between 0 and H
v_{des}	is the building orthogonal design wind speed determined at the height H

$$E_t = \frac{\pi N}{\left(1 + 70.8 N^2\right)^{5/6}} \tag{3.18}$$

where

N	is the reduced frequency as $n_a L_H[1 + (g_v I_H)]/v_{des}$
b_{0H}	is the average breadth of the structure between 0 and H

According to the AS/NZS the peak along-wind direction acceleration at the top of the structure can now be calculated using the following equation:

$$a_{x.max} = M_0 H^2 \times \text{resonant component of peak base bending moment} \quad (3.19)$$

$$= \frac{3}{M_0 H^2} \frac{\rho_{air} g_R I_H \sqrt{\frac{SE_t}{\xi}}}{(1 + 2g_v I_{H)}} \left\{ C_w \sum_{z=0}^{h} [v_{des}(z)]^2 B_z - C_l [v_{des}(H)]^2 \sum_{z=0}^{h} B_z z \Delta z \right\}$$

where M_0 is the average mass per unit height in kg/m
ρ_{air} is the density of air (1.2 kg/m^3)
$v(z)$ is the orthogonal design wind speed of height z
$v(H)$ is the orthogonal design wind speed evaluated at height H
B_z is the average breadth at section at height z
Δz is the height of the section of the structure upon which the wind pressure acts
C_w aerodynamic shape factor in windward direction (building side against the wind direction)
C_l aerodynamic shape factor in leeward direction (opposite building side from the wind)

3.4 Cross-wind accelerations

The peak crosswind acceleration and combination with the along-wind direction has been covered currently at least in two designing standards which are the AIJ Recommendation and the AS/NZ Standard. The AIJ also provides a calculation method for the torsional direction.

AIJ-RLB-2015

According to AIJ, the maximum acceleration $a_{L.max}$ at the top of a tall building in the cross-wind direction is calculated by the equation (x.x). The process is very similar to the along-wind direction, but the formulas are slightly different. Also all the values are considered in the cross-wind direction.

$$a_{L.max} = \frac{q_H g_L B H C'_L \lambda \sqrt{R_L}}{M_L} \quad (3.20)$$

where q_H is the design velocity pressure as $\frac{1}{2}\rho U_H^2$ where ρ is the air density (assumed to be 1.22) and U_H the design wind speed
g_L is the peak factor for the cross-wind vibration as (3.21)
B is the width of the building
H is the mean roof height of the building
C'_L is the rms overturning moment coefficient as defined in (3.22)

	λ	is the mode correction factor of generalized wind force as defined in (3.8)
	R_L	is the resonance factor as (3.23)
	M_L	is the generalized mass of the building for cross-wind vibration as (3.14)

$$g_L = \sqrt{2\ln(600 f_L) + 1.2} \tag{3.21}$$

where f_L is the first mode natural frequency of vibration of a structure in the crosswind direction

The rms overturning moment coefficient in the cross-wind direction is defined by the equation (3.22). The value is calculated from the shape of the building. Unlike in the along-wind direction, it does not account the building height.

$$C'_L = 0.0082(D/B)^3 - 0.071(D/B)^3 + 0.22(D/B) \tag{3.22}$$

where D is the depth of the building
B is the width of the building

The resonance factor for cross-wind is calculated by the equation (x.x). The biggest difference in the formulas compared to the along-wind direction is in the spectral coefficient, which had to be calculated using more complicated method to represent the cross-wind direction. It is determined by using the structural factors β_1, β_2, f_{s1} and f_{s2}.

$$R_L = \frac{\pi F_L}{4\varsigma_L} \tag{3.23}$$

where F_L is the spectral coefficient of overturning moment in cross-wind direction defined by (x.x)
ς_L is the damping factor

$$F_L = \sum_{j=1}^{m} \frac{4\kappa_j(1 + 0.6\beta_j)\beta_j}{\pi} \frac{(f_L/f_{sj})^2}{\left\{1 - (f_L/f_{sj})^2\right\}^2 + 4\beta_j^2 (f_L/f_{sj})^2} \tag{3.24}$$

where m is 1 if $D/B < 3$ and 2 if $D/B \geq 3$ where D is the breadth of the building and B is the width of the building
κ_1 is 0.85
κ_2 is 0.02
β_1 is the factor defined by (3.25)
β_2 is the factor defined by (3.26)
f_{s1} is the factor defined by (3.27)
f_{s2} is the factor defined by (3.28)

$$\beta_1 = \frac{(D/B)^4 + 2.3(D/B)^2}{2.4(D/B)^4 - 9.2(D/B)^3 + 18(D/B)^2 + 9.5(D/B) - 0.15} + \frac{0.12}{(D/B)} \tag{3.25}$$

$$\beta_2 = \frac{0.28}{(D/B)^{0.34}} \tag{3.26}$$

$$f_{s1} = \frac{0.28}{\{1 + 0.38(D/B)^2\}^{0.34}} \frac{U_H}{B} \tag{3.27}$$

$$f_{s2} = \frac{0.56}{(D/B)^{0.85}} \frac{U_H}{B} \tag{3.28}$$

Lastly, the generalized mass of the building for cross-wind vibration M_L is calculated using the same equation (3.14) as for the along-wind direction, but the mode shape μ of the building is now also considered in the cross-wind direction.

AS/NZS 1170:2-2002

The peak acceleration at the top of the tall building in the crosswind direction is determined as:

$$a_{y.max} = \frac{1.5Bg_R}{M_0}\left[\frac{0.5\rho_{air}v_{des}^2}{(1+g_vI_H)^2}\right]K_m\sqrt{\frac{\pi C_{fs}}{\xi}} \tag{3.29}$$

where	B	is the breadth of the structure, normal to the wind stream
	g_R	is the peak factor for resonant response given by (3.30)
	M_0	is the average mass per unit height in kg/m
	ρ_{air}	is the density of air (1.2 kg/m^3)
	g_v	is the peak factor for the upwind velocity fluctuations, may taken as 3.7
	v_{des}	is the design wind speed
	I_H	is the turbulence intensity, obtained from a table
	K_m	is the mode shape correction factor for crosswind acceleration given by (3.31)
	ξ	ratio of structural damping to critical damping of a structure, obtained from a table

$$g_R = \sqrt{[2 \times \ln(600n_c)]} \tag{3.30}$$

where	n_c	is the first mode natural frequency of vibration of a structure in the crosswind direction, in Hertz

$$K_m = 0.76 + 0.24k \tag{3.31}$$

where	k	is the mode shape power exponent for the fundamental mode values k should have are between 0.5 and 2.3: 0.5 for a slender framed, moment resisting structure 1.0 for a building with central core and moment resisting façade 2.3 for a tower decreasing in stiffness with height, or with large mass at the top

The exponent k can also be obtained from fitting:

$$\phi_1(z) = \left(\frac{z}{H}\right)^k \tag{3.32}$$

where	$\phi_1(z)$	is the first mode shape as a function of height z

The crosswind force spectrum coefficient C_{fs} is calculated from reduced velocity V_n, which uses the design wind speed v_{des} as:

$$V_n = \frac{v_{des}}{n_c B(1 + g_v I_H)} \tag{3.33}$$

and it varies depending on the cross section and turbulence intensity of the structure.

3.5 Torsional accelerations

AIJ Recommendation is currently one of the only structural designing codes that cover the peak accelerations for torsional direction. The method covered here is from the AIJ-RLB-2015 version.

The maximum torsional response acceleration $a_{T.max}$ at the top of the building is given by equation (3.34). The acceleration is the angular acceleration (rad/s^2) which differentiates it from the other two directions. The maximum acceleration caused by torsion (m/s^2) can be calculated by multiplying $a_{T.max}$ with the distance from the elastic centre of the structure.

$$a_{T.max} = \frac{0.6 q_H g_T B^2 H C'_T \lambda \sqrt{R_T}}{I_T} \tag{3.34}$$

where
- q_H is the design velocity pressure as $\frac{1}{2}\rho U_H^2$ where ρ is the air density (assumed to be 1.22) and U_H the design wind speed
- g_T is the peak factor for the torsional vibration as (3.35)
- B is the width of the building
- H is the mean roof height of the building
- C'_T is the rms overturning moment coefficient as defined in (3.36)
- λ is the mode correction factor of generalized wind force as defined in (3.8)
- R_L is the resonance factor as (3.37)
- M_L is the generalized mass of the building for cross-wind vibration as (3.14)
- I_T is the generalized mass of building torsional vibration defined in (3.44)

The peak factor for the torsional direction is calculated using the same equations as for the along-wind and cross-wind directions, but the natural frequency is substituted by the torsional direction.

$$g_T = \sqrt{2\ln\left(600 f_T\right) + 1.2} \tag{3.35}$$

where f_T is the first mode natural frequency of vibration of a structure in torsional direction

The rms overturning moment in the torsional direction is defined by the following equation:

$$C'_T = 0.04\left(\frac{D}{B}\right)^2 + 0.02 \tag{3.36}$$

The equation for the resonance factor R_T is the same as in the two previous direction. However, the biggest difference is again in calculating the spectral coefficient F_T, which is required to determine the resonance factor. Since torsional vibration is a complex phenomenon, many different factors are required to bring it on a standardized level. AIJ Recommendations provides a table with the approximated values for the parameters of the spectral coefficient, which depend on the relation of the building width and depth.

$$R_T = \frac{\pi F_T}{4\varsigma_T} \tag{3.37}$$

where F_T is the spectral coefficient of torsional moment

ς_T is the damping factor

$$F_T = 0.8F_B + v_1 F_V + w_1 F_w \tag{3.38}$$

The needed non-dimensional parameters are calculated using the following approximate equations:

$$F_B = \frac{18 f_m^*}{\{1 + 0.46(18 f_m^*)^{1.8}\}^{2.3}} \tag{3.39}$$

$$F_V = \frac{1}{v_2\sqrt{2\pi}}^{-0.5\left(\frac{\ln(f_s^*/v_3)+0.5v_2^2}{v_2}\right)^2} \quad (3.40)$$

$$F_w = \frac{1}{w_2\sqrt{2\pi}}^{-0.5\left(\frac{\ln(f_m^*/w_3)+0.5w_2^2}{w_2}\right)^2} \quad (3.41)$$

$$f_s^* = \frac{8.3 f_T B\{1 + 0.38(D/B)^{1.5}\}^{0.89}}{U_H} \quad (3.42)$$

$$f_m^* = \frac{f_T B}{U_H} \quad (3.43)$$

where v_1, v_2, w_1, w_2, w_3, v_3, are parameters from a table in the standard

Lastly, the generalized mass of building for torsional vibration I_T is defined with the following equations:

$$I_T = \int_0^H i(Z)\mu^2(Z)dZ \quad (3.44)$$

where $i(Z)$ is the inertial moment per unit height at height Z defined by (3.45)

μ is the first mode shape of building for torsional vibration

$$i(Z) = m(Z)(B^2 + D^2)/12 \quad (3.45)$$

where $m(Z)$ is the mass per unit height at height Z

3.6 Combining the dynamic components

AS/NZS 1170.2-2002

The Australian/New Zealand Standard gives one equation for the combined peak scalar dynamic action effect ε_t as:

$$\varepsilon_t = \varepsilon_{a,m} + \left[\left(\varepsilon_{a,p} - \varepsilon_{a,m}\right)^2 + \varepsilon_{c,p}^2\right]^{0.5} \tag{3.46}$$

where $\varepsilon_{a,m}$ is the action effect derived from the mean along-wind response by (3.47)
$\varepsilon_{a,p}$ is the action effect from the peak along-wind response
$\varepsilon_{c,p}$ is the action effect from the peak crosswind response

$$\varepsilon_{a,m} = \frac{\varepsilon_{a,p}}{\left[C_{dyn}(1 + 2g_v I_H)\right]} \tag{3.47}$$

where C_{dyn} is the dynamic response factor by (3.15)
$\varepsilon_{a,p}$ is the action effect from the peak along-wind response
$\varepsilon_{c,p}$ is the action effect from the peak crosswind response

The AIJ-RLB-2015 does not give any guidance for combining the peak accelerations for different response directions. It does provide a guide for combining the wind loads in different load cases for each direction of the wind, which are not discussed in this paper.

3.7 Previous studies comparing different standards

Since all standards calculate the accelerations slightly differently, some comparisons have been made throughout the years to both along-wind and crosswind directions. Zdraveski and Mickoski tested in the year 2015 [11] how the dynamic acceleration response calculated using the EN1991-1-4, ASCE7 and AIJ guidelines compared with 25 years of full-scale monitoring of a 14-story reinforced concrete office. The geometry of the building is complex with changing floor plan. Wind data was collected every three hours and it contained information about the mean wind direction and wind velocity of the first 10 minutes at 10-meter height as well as the maximum 10-minute mean velocity and maximum gust wind velocity. [11]

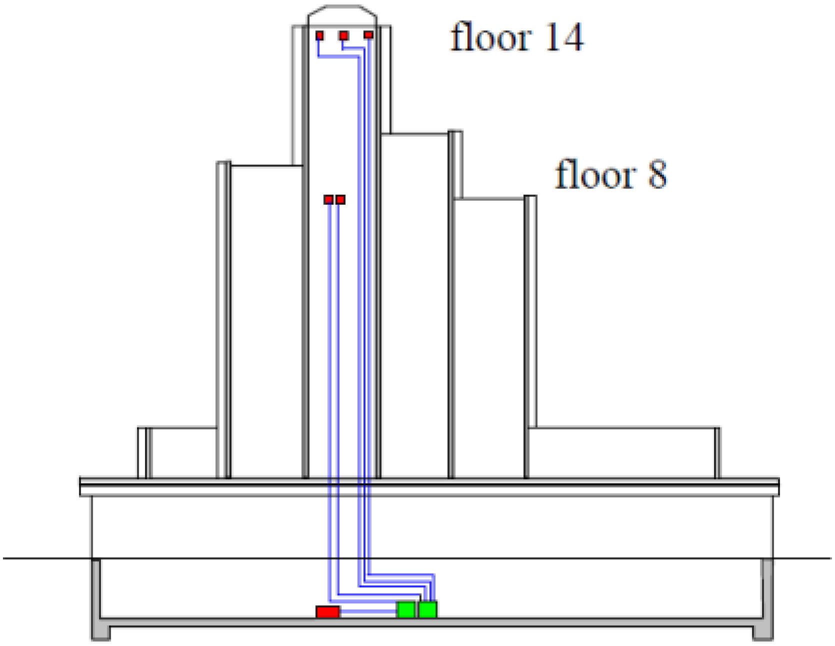

Figure 4. *A 14-storey reinforced concrete building tested by Zdraveski and Mickoski [11]*

The collected data about acceleration in both along and crosswind directions were divided in two groups. One contained the wind in east-west directional sector and the other group contained the wind data within north-south sector. These were the most sensitive direction for the building, since the wind direction was perpendicular to both cross sections of the building. Although the strongest storms commonly acted in the S-W direction, most of the storms that triggered the monitoring systems were in the E-W directions. This was since the E-W direction of the building had weaker axis of the top floors and was more prone for acceleration in this direction. [11]

The mode shape exponent for calculating the accelerations were estimated using a finite element model and the results were different for each direction of vibration. The mode shape in the E-W direction had the exponent of 2 while in the N-S direction it resembled more a linear shape with mode shape exponent of 1. These mode shapes were then used for calculating the accelerations with all three design codes. [11]

What was discovered, was that the results calculated using the three different design codes varied quite considerably. All three codes gave stronger along wind response for the E-W direction than in the N-S direction, which was consistent with the recorded results. Since the AIJ recommendations provide method to calculate the across wind acceleration as well, it was also compared in the calculations. The Eurocode predicted the greatest along wind responses while the AIJ recommendation predicted the lowest response. The results given by the Eurocode were too high on average compared to the monitored data, but there were few points where the recorded accelerations grew even higher than that. One notable issue with the Eurocodes results were that it predicted the accelerations to grow rapidly with increased wind velocity. It leads to question, if the results for higher wind speeds provide very accurate results. The ASCE procedure predicted lower values than the Eurocode which were quite close to the average recorded along wind responses. The AIJ gave the lowest results for the along wind directions which were clearly lower than the recorded data. Then again, the across wind accelerations calculated using the AIJ recommendations compared relatively well with the recorded data, as seen in the Fig. 5 and Fig. 6. [11]

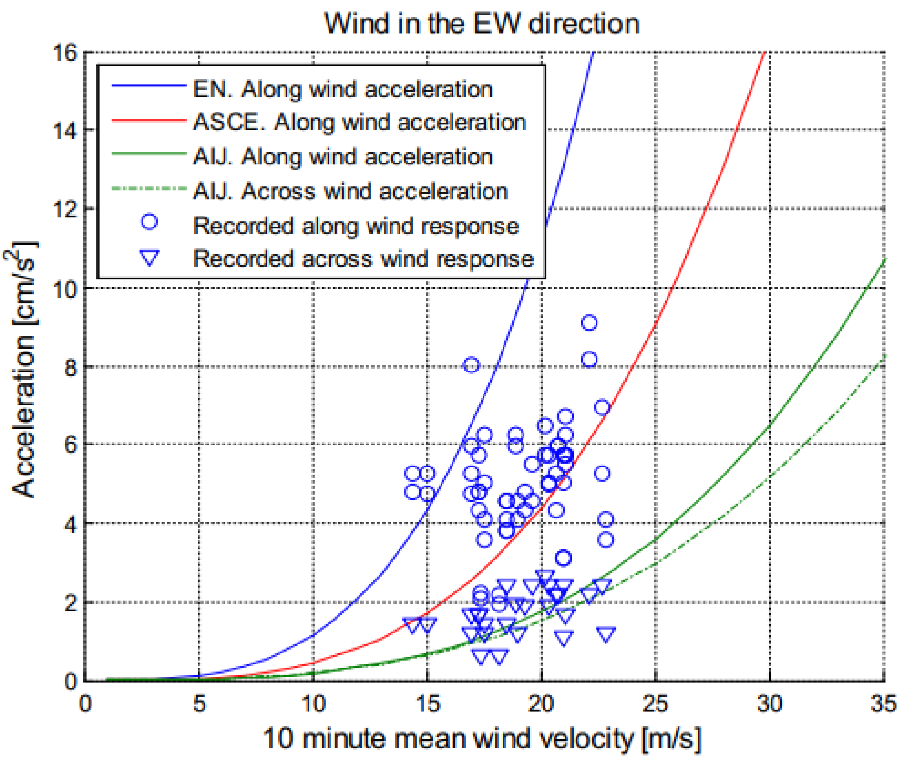

Figure 5. *Results of the 14-storey reinforced building in E-W direction of the wind for along-wind and cross-wind accelerations [11]*

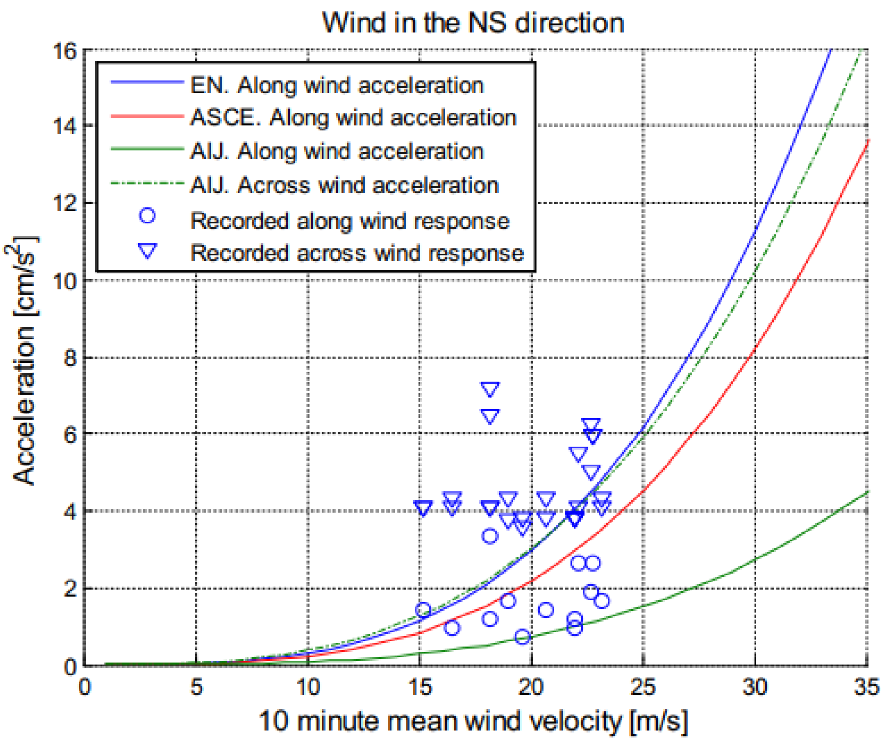

Figure 6. *Results of the 14-storey reinforced building in N-S direction of the wind for along-wind and cross-wind accelerations [11]*

According to this study, the biggest difference between the design code procedures seemed to be caused by the response factor R^2 which is calculated based on the modal parameters of the structure. [11]

The results show, that for this specific building, the across wind responses were around 3 times higher in the N-S direction compared to the along wind responses. They were also significantly higher than all the results provided by the designing codes, although the AIJ across wind responses were again quite accurate. Also, the Eurocodes results for along wind responses were closer to the recorded across wind data in the N-S direction than to the along wind response data. The differences in the response direction were caused by the geometry of the building, which was stiffer in the E-W direction and therefore led to stronger responses in the across wind section when the wind was blowing from the N-S direction. [11]

Zdraveski and Mickoski mentioned in the study, that the AIJ recommendations had provided very accurate results for higher buildings in the past, which led them to believe, that it was more accurate for buildings with natural frequencies below 1 Hz. The building in this study was stiffer, which might have been one factor for the poor performance of the results calculated using the AIJ recommendation. It is important to note however, that in reality the stiffness of the building chanced due to the varying floor plan and the simplified methods provided by the design codes were not completely trustworthy when estimating the stiffness of the structure in general. [11]

Another study was carried out by Holmes in 2013 [21] where he compared along-wind and cross-wind base moments for a generic tall building between wind tunnel tests and three standards, which were the ASCE 7 (2010), AS/NZS (2011) and the Hong Kong Code of Practice (2004). He used a new high-frequency base balance (HFBB) wind tunnel test data which is discussed more in the chapter 4. [21]

The study focused on two building, of which the first one was referred as the 'Basic' building, with 180 meters in height and rectangular 30 m x 45 m cross section. The second building was referred as the 'Advanced' building, of which height (240 meters) put it outside of the scope of the wind loading standards. However, the results were still used as a comparison with the standards and the basic building. [21]

Three uncoupled dynamic modes were defined in the basic building, with the sway frequencies of 0.20 Hz and 0.23 Hz. The structural damping responses was limited to 1.0% and critical damping to 2.5%, which represent serviceability and ultimate limit state conditions. The building was assumed to be located in urban terrain. [21]

As a result, the AS/NZS provided the closest outcome to the averages of the wind tunnel data in all along-wind cases. The difference was from -4% to +8%, where +8% meant slightly higher predictions for the along wind base moment than the wind tunnel data. The second closest results were provided by the ASCE, which were 8-17% below the average values. Clearly lowest results were from the Hong Kong Standard, which were 27-33% below the wind tunnel data. However, these low results were due to too low drag coefficient for the cross section, not because of errors in the gust response factor formulation. [21]

The difference between ASCE and AS/NZS was noted to be likely due to the difference in calculating the gust effect factor. ASCE and AS/NZS uses different duration gust and in the expression different factor is used to account for the reduced peak factor. According to the research, in ASCE this factor is 1.7 in the denominator in the following equation:

$$G_f = 0.925\left(\frac{1 + 1.7I_z\sqrt{g_Q^2Q^2 + g_R^2R^2}}{1 + 1.7g_vI_z}\right) \tag{3.48}$$

If the factor in the denominator would be changed from 1.7 to 2.0, it would be equal to the AS/NZS expression, and also it would make the ASCE results differ only ±9% from the wind tunnel data in this test and therefore make a significant improvement. [21]

Finally, the combined results for accelerations in both along-wind and cross-wind directions were compared with the AS/NZS and the results were quite conservative in both 0- and 90-degree wind direction. The standard predicted the acceleration to be about 30% higher than the wind tunnel data. [21] It should be noted that ideally the standard predictions are better to be higher than lower compared to the actual situation, since there should be room for safety factors in the standardized methods.

3.8 Comparison between calculation parameters in standards

As the previous chapters show, design standards treat wind-induced vibration differently and use varying criteria for the designing process. In this chapter some of the key differences in the basic calculation parameters are concluded and compared.

First area of variance is the way to determine the design wind velocity. ASCE and AS/NZS both use the 3-s gust wind speed as the basic wind speed for calculation, which translates to higher wind speed values compared to the 10-minute mean wind speed used in other design standards, such as SFS-EN 1991-1-4 and AIJ Recommendation for Loads. The basic values of wind speeds vary greatly according to the geographical location of interest.

Peak accelerations are usually determined for serviceability design and the time period used in designing varies. The longer the period, often referred as the return period of the wind, the higher is the probability for heavy wind conditions to occur. Also, the averaging time for vibration affects the final peak acceleration results. Averaging time is needed to define the peak factor, that is used in most standards to calculate the peak acceleration of vibration. Longer averaging time increases the values of peak accelerations.

Then, a notable difference in the basic values is the reference height of the structure. Many design standards, like AS/NZS and AIJ determine the peak accelerations at the top of the building, whereas other standards like ASCE and SFS-EN 1991-1-4 uses 60% of the maximum height. These differences are combined in the Table 1.

Table 1. Differences in the calculation basis according to different design standards

	ASCE	AS/NZS	AIJ	EN	ISO
Basic wind velocity	3 s	3 s	10 min	10 min	3 s / 10 min
Averaging time for vibration	1 h	10 min	10 min	10 min	10 min
Reference height	0.6H	H	H	0.6H	H
Return period for serviceability design	10 years	10 years	100 years	50 years	1 year

Kwon and Kareem have carried out an extensive comparison [29] between different design standards and conclude, that if small modifications are made to the basic calculation parameters and velocity profiles, the differences between standards can be eliminated to significant degree. Table 2. shows the terrain/exposure category comparison according to their research for the design standards discussed in this paper. This comparison is later utilized in calculations in the chapter 6.

Table 2. Terrain category comparison according to Kwon & Kareem [29]

ASCE	AS/NZS	AIJ	EN	ISO
-	4	V	-	4
A	-	IV	IV	-
B	3	III	III	3
C	2	II	II	2
D	1	I	I	1
-	-	-	0	-

When the basic calculation parameters are modified, different design standards seem to give relatively coherent results in the along-wind direction, that is based on the gust loading factor approach. In the across-wind and torsional directions more variation can be detected, since these directions are more affected by the vortex and wake-induced effects. [29]

4. OTHER ESTIMATION METHODS FOR DYNAMIC RESPONSES OF TALL BUILDINGS

Updating designing standards and codes is a slow process and new calculation methods often take years to be implemented in standardized level. However, in structural designing field, serviceable methods to evaluate more advanced structures are required. Different simplified estimation methods are useful tools, and for more advanced cases computer analysis and wind tunnel tests are used.

Especially the wind-induced torsional vibration for tall and slender buildings is not covered well on standardize level, as seen on the previous chapter. For that reason, in this chapter some other methods to evaluate structures are discussed.

4.1 NatHaz aerodynamic loads database

The NatHaz Aerodynamic Loads Database (NALD) was established in the year 2000 and it is an online experimental database that provides users access to wind tunnel test data and provides guides to determinate dynamic responses in all three directions. The database has been noted by ASCE 7-05 (C6.5.8) to serve as an alternative method of estimating the dynamic wind load effects for high-rise buildings. The database can be found from: http://aerodata.ce.nd.edu/ [12]

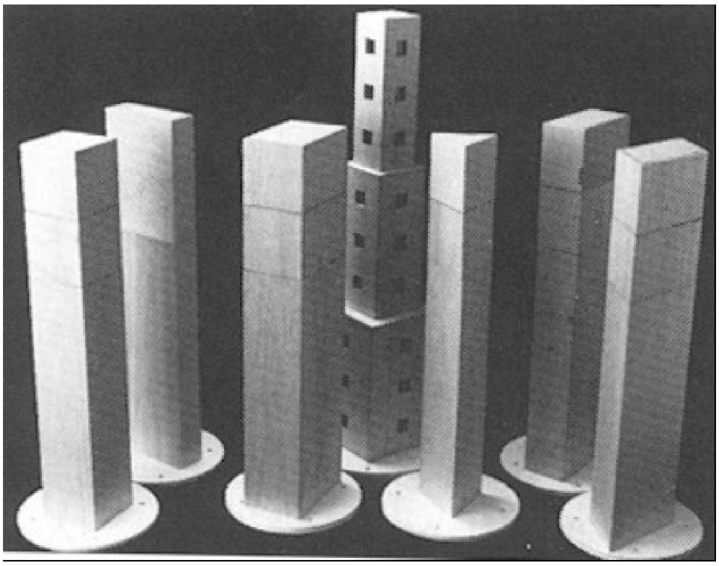

Figure 7. *Balsa Wood Models [12]*

The NALD has cumulated results from 162 different tests done to balsa wood models, shown in Fig. 7. The models were tested in a boundary level wind tunnel and turbulent boundary levels that were generated by the natural action of surface roughness added on the tunnel floor and upstream spires. The tests include nine cross-sectional shapes, three model heights, two exposure categories and three response directions (along-wind, cross-wind and torsional). The results have been compared with previous tests and models, and in most cases the results have been comparable. [13]

On the database the user can select a structure from available shapes, heights and conditions and get the information about the dynamic responses in any of the three directions. The user interface is shown in the Fig. 8. [13]

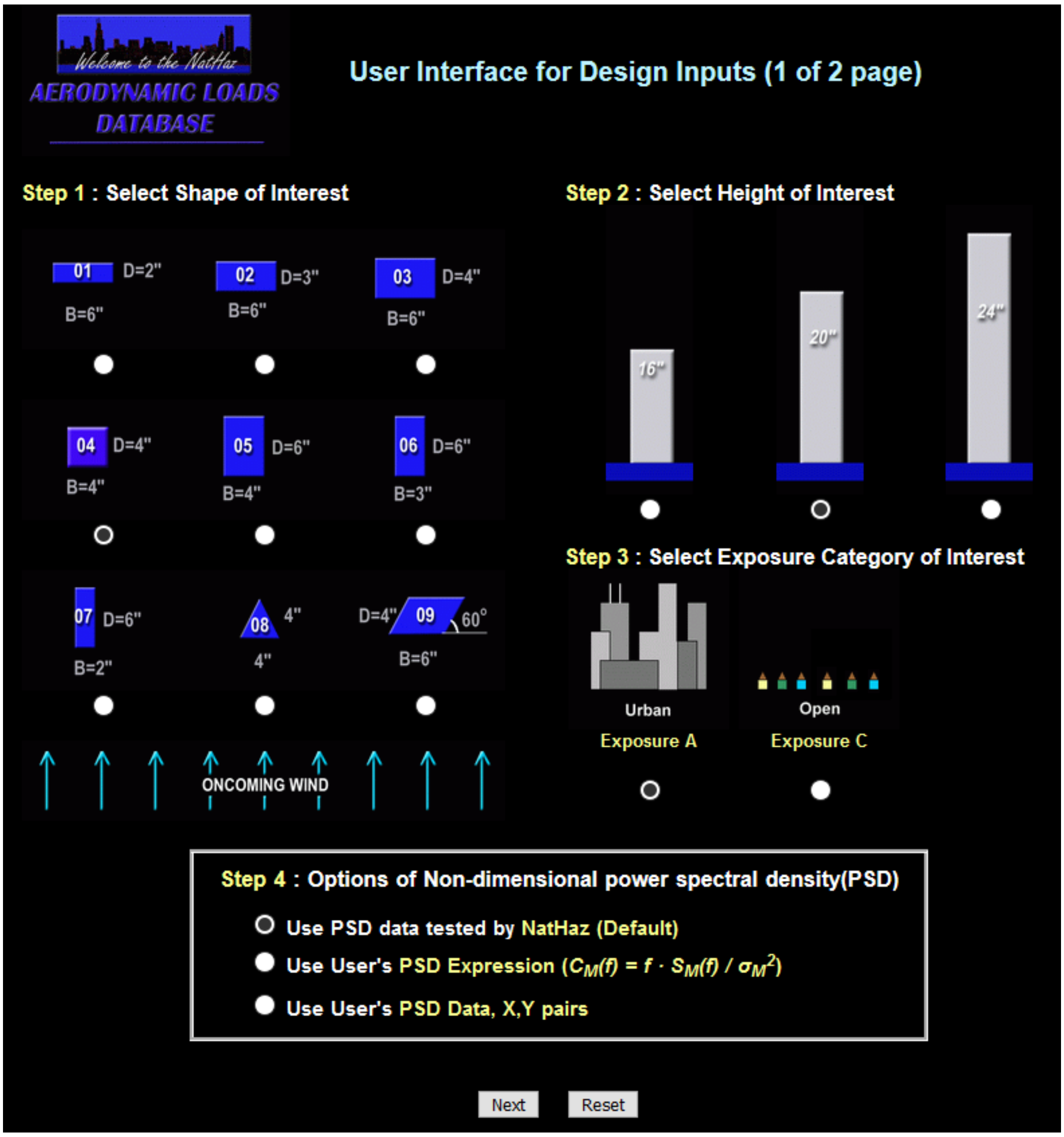

Figure 8. *User interface of NatHaz Aerodynamic Loads Database [12]*

After selecting the initial characteristics for the building of interest, the user can now insert further information of the building that is required in the calculation process. These factors are shown in the Fig. 9. The calculations require natural frequencies of the building, the basic geometry, the density, damping ratio and force coefficient. Then the basic wind speed of the 3-s gust wind according to ASCE standard is required. [12]

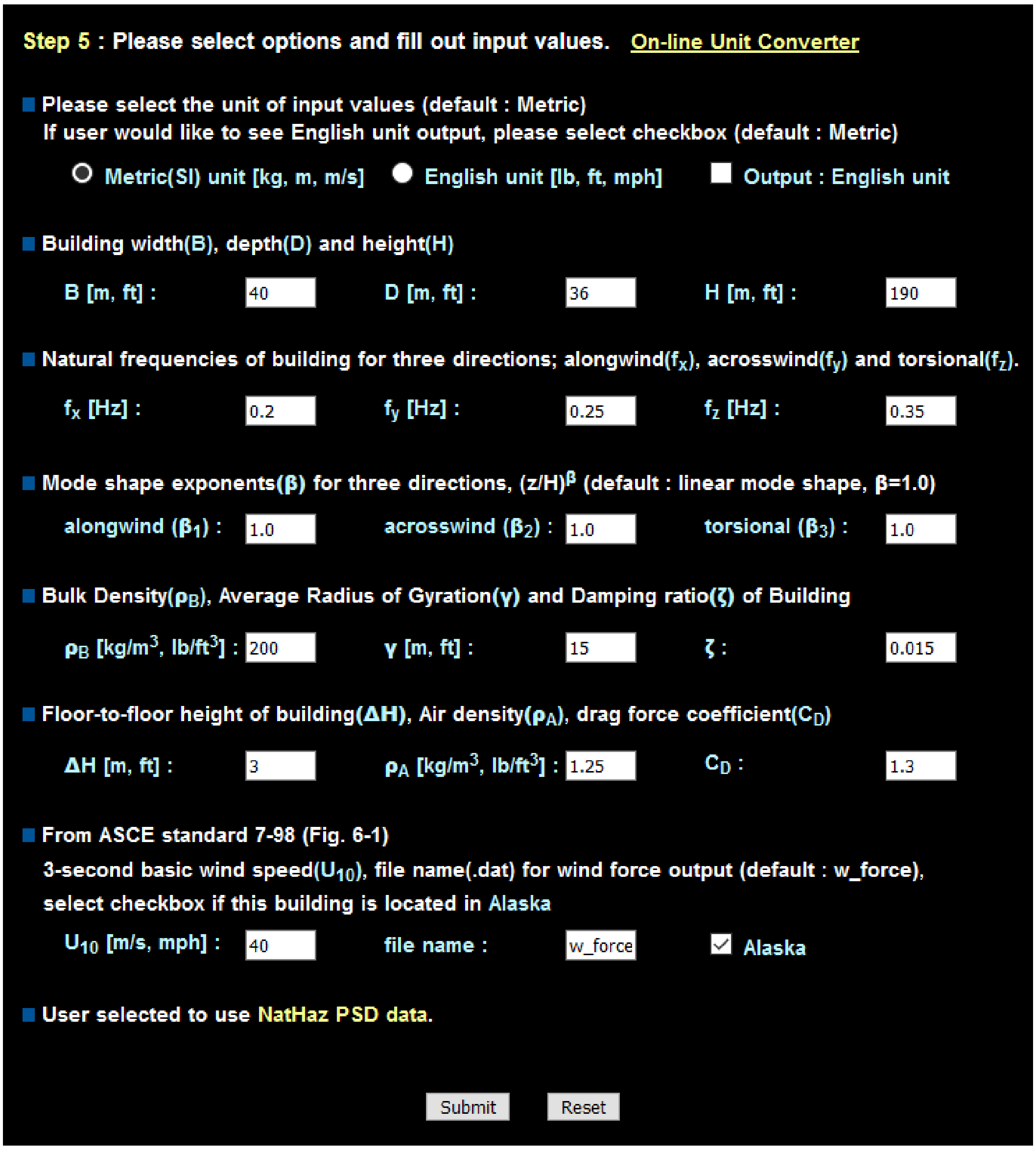

Figure 9. *User interface to add calculation values on NatHaz Aerodynamic Loads Database [12]*

The values shown in the Fig. 9 are used as an example to showcase the results the database provides. One of the most helpful calculation parameters that are obtained from the database are the base bending moment coefficients, which are usually difficult to calculate and specially to evaluate the reliability of the calculated results. The database uses a newer gust loading factor (GFL) format that is proposed by Zhou and

Kareem in 2001 [18]. [13] GFL essentially describes the ratio between the extreme and the mean displacement response and characterizes the dynamic properties of the wind in calculations. It is used to determine the equivalent static wind loading that is used as a designing basis for wind loading in most building codes. [18]

The new method combines the base bending moment with the standard GLF which are then distributed to each floor in terms of moment by the proposed GLF. This approach is similar to the method used in earthquake engineering, where the mean base shear is distributed to all floors. It has provided more accurate results for tall buildings compared with the previous methods of the GFL which are based on the displacements. [18] By using the aerodynamic base bending moment and base torque, the wind-induced response of the structure can be calculated using a random vibration analysis. In this approach, the mode shape correction might not be necessary, and it has proven to generate accurate results for the along-wind direction. [13] The RMS base bending moments for the example values is shown in the Fig. 10. The database also provides equivalent static wind loads for survivability design and maximum accelerations for serviceability design.

1-hour mean wind speeds for designs

Survivability design (50-year return period) : U_H = 32.02 *m/s*

Serviceability design (10-year return period) : U_H = 27.86 *m/s*

RMS base moment coefficients(σ_{CM}), reduced frequencies($f_1 \cdot B / U_H$) and non-dimensional moment coefficients($C_M\ (f_1)$)

	σ_{CM}	$f_1 \cdot B / U_H$		$C_M\ (f_1)$	
		50-year	10-year	50-year	10-year
Alongwind	**0.109**	**0.250**	**0.287**	**0.030**	**0.022**
Acrosswind	**0.133**	**0.312**	**0.359**	**0.016**	**0.010**
Torsional	**0.044**	**0.437**	**0.503**	**0.043**	**0.043**

Figure 10. *Design wind speeds and RMS base bending moment coefficients according to NALD [12]*

The database provides 10-year and peak lateral accelerations and corresponding lateral torsional accelerations in all three response directions. All the displacements and accelerations are calculated on the roof level of the structure. The system is designed to reduce human errors and provide alternative options for curve-fit expressions provided in designing codes, that often generalize the phenomena. [13] The results for the example values for serviceability design are showcased in the Fig. 11. The database provides acceleration results in g-units, which is converted to m/s^2 as:

$$1\ g = 9.80665\ m/s^2$$

Serviceability Design (10-year wind) : Peak and RMS Accelerations

	Peak Accelerations at roof	
Alongwind	**9.17** *milli-g*	
Acrosswind	**6.89** *milli-g*	
Lateral Accelerations at Corner Induced by Torsion	**0.00554** *rad/s*^{2}	Alongwind component : **11.30** *milli-g*
		Acrosswind component : **10.17** *milli-g*
Total Lateral Accelerations at Corner	Alongwind component : **14.55** *milli-g* Acrosswind component : **12.28** *milli-g*	

	RMS Accelerations at roof	
Alongwind	**2.42** *milli-g*	
Acrosswind	**1.79** *milli-g*	
Lateral Accelerations at Corner Induced by Torsion	**0.00141** *rad/s*^{2}	Alongwind component : **2.88** *milli-g*
		Acrosswind component : **2.59** *milli-g*
Total Lateral Accelerations at Corner	Alongwind component : **3.76** *milli-g* Acrosswind component : **3.15** *milli-g*	

Figure 11. *Results for serviceability design according to NALD [12]*

The values of the database work as a great comparison for any calculated data from a model or as a quick way to determine basic knowledge of the structure in very early designing process. In working conditions, it is often enough to be able to determine the possibility of designing a structure within a realistic framework.

4.2 Wind tunnel testing

As mentioned earlier, the building codes are generally developed for low-rise buildings and provide very limited methods to calculate the wind phenomena for high-rise buildings. For that reason, wind tunnel tests are still considered to be the best method for determining the building motions caused by the wind. [23] However, selecting an appropriate wind tunnel test method is important, since some tend to work well for the sway motions in along-wind and cross-wind directions, but does not resolve the complicated nature of torsional response [7]. A wind tunnel test conditions are demonstrated in the Fig. 12.

Figure 12. *Wind tunnel testing model [23]*

Since wind tunnel tests are rather complicated and expensive experiments in regular designing procedure, it is important to have some knowledge of when the information they provide is valuable and required. Irwin et al. [23] have stated the following criterion of when a wind tunnel test could be an advisable solution for a designed structure:

1. The height of the building is over 120 meters.
2. Four times the average width of the building is less than the building height.
3. The lowest natural frequency of the building is less than 0.25 Hz.
4. The reduced wind velocity $v/_{fB}$ is more than 5, where v is the mean hourly wind velocity evaluated at the top of the building, f is the lowest natural frequency and B is the average width of the building. [23]

After determining the need for a wind tunnel test, is then an appropriate method to be selected. A widely used technique for determining the wind-induced vibration for advanced structures is the high-frequency force balance (HFFB) method, which has replaced aeroelastic models in wind-induced response analysis. In this method, a rigid model is used, which is supported at the base by a measuring system that can detect the mean and fluctuating wind forces and moments to a high frequency. [1] However, determining torsional response using this method has not given as accurate results as the along-wind and cross-wind sway responses, since the model does not resolve generalized loads for torsional motion. Generalized loads for torsion on normalized mode shapes have found to be between 50% and 75% of the base torque for an ideal building, which might not be an accurate enough result for a torsionally sensitive building. [7] Because of this a modified HFFB method has been developed by Xie and Irwin [25] called the multi-high-frequency force balance technique (MHFFB). This method generates generalized torsional loads by assuming that the background pressure distribution for torsional loads works geometrically the same way as it does for horizontal loads. The MHFFB method has found to provide more precise results and is a good wind tunnel method for both torsional and sway responses. [25]

Alternative method for the HFFB method is a high-frequency pressure integration method (HFPI), which is using numerous pressure measurements all over the building surface. It can determine the generalized loads by on-line pressure integration by simultaneously measuring pressure at various locations and theoretically it should provide more accurate results especially in the torsional direction. The method still has some limitations for buildings with complicated exterior surfaces, because it can create problems for the pressure taps. [7]

A further improvement of the HFFB method is the multi force balance method (MFB). MFB model works well to determine torsional measurements for torsionally sensitive buildings and its mechanism is based on mounting the structure on a multi force balance system, that vertically splits the structure into several substructures. The torsional load measures provided are on the same level of completeness as the horizontal loads. [7]

The methods mentioned here are combined in the Table 3. according to the circumstances of the designed structure.

Table 3. The best wind tunnel test methods for torsional response according to Xie & Irwin [7]

<table>
<tr><th colspan="2">Circumstances</th><th>Preferred Method</th></tr>
<tr><td colspan="2">Building exterior surfaces are smooth and simple</td><td>HFPI</td></tr>
<tr><td rowspan="2">Building exterior surfaces are complicated with many architectural details</td><td>Ratio of building width/height is high</td><td>MFB</td></tr>
<tr><td>Ratio of building width/height is low (slender buildings)</td><td>HFFB</td></tr>
</table>

It is also to be noted, that being able to understand the results from a wind tunnel test requires proper judgement and expertise on the field in general, as well as understanding the difference between different testing methods and laboratories. If the along-wind responses are the main concern for a building, wind tunnel results are likely to be quite similar to those calculated using building codes. For very tall and slender buildings, the cross-wind responses are often dominant and since this direction is more sensitive to the shape of the building, the results calculated using building codes might be rather different from the results provided by wind tunnel testing. If the cross-wind or torsional responses are the main concern, selecting an appropriate testing method that focuses on the shape of the building is important. It is also important to note, that sometimes stiffening a building might increase the cross-wind response, whereas it is a general method for reducing the along-wind responses. [23]

5. HUMAN PERCEPTION OF WIND-INDUCED BUILDING MOTION

While all three components of wind-induced vibration might, in some extreme cases, cause structure failures and are important to carefully investigate in the designing process, often the biggest limitation for the sway and twist motions of tall buildings ends up being the occupants' comfort. Studies have shown that excessive vibration may cause difficulties to perform manual tasks, reduce work performance, cause sleepiness and even induce motion sickness. [26]

Building codes are generally promoting safety issues rather than issues relating to people's comfort. For that reason, many major designing codes do not provide clear limits for the serviceability criteria of wind-induced building motion, which is why different publications need to be referenced. [27] In this chapter, some of the current criteria are evaluated and compared.

5.1 ISO 10137

Humans respond mainly to fluctuating and peak accelerations of a building, which are the limits most often referred when measuring building motion. The most generally accepted criterion to measure human perception of building motion is given in Annex D of International Organization for Standardization (ISO) 10137 in 2007, which is showcased in the Fig 13. [1, s. 242]

The ISO criteria measures the peak values at 1-year return period. It presents a separate curve for residential and office buildings, of which the former requires 2/3 times lower frequencies. The frequency stands the first natural frequency of either the structural direction of a building or the torsion of building in Hz. [27]

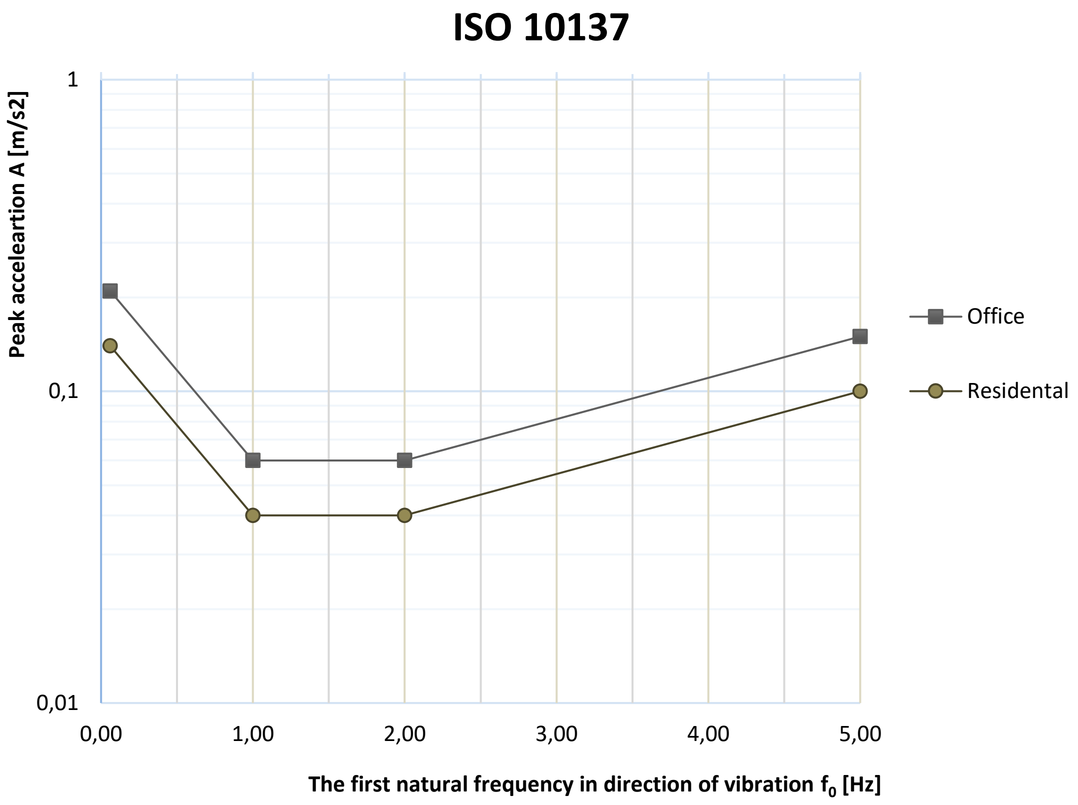

Figure 13. *ISO 10137 standard on building serviceability*

ISO 10137 curve is frequency based, since studies have shown that people are more sensitive to motion up to 1 Hz. However, frequency and acceleration are able to identify the building motion only on one point in time, and do not address duration effects. Mild motion sickness, described as sopite syndrome is one effect of long-term exposure to gentle accelerations, which are often overlooked in designing process, partly because of the complex relationship between accelerations, duration and frequency. [26]

The difference between office and residential buildings is explained due to the fact that office buildings are occupied for lower proportions of time than residential buildings and they are also often shut down during severe wind phenomena, such as typhoons. The difference has also reserved critique, since office buildings are primarily designed to support sedentary work at a desk, which leads to more obvious sensing of building motion than wider range of activities in residential buildings. [26]

5.2 AIJ-GBV-2004

Another guideline for building motion evaluation is published by The Architectural Institute of Japan in the AIJ-GBV-2004. The guideline has been calibrated to research made in Japan of the occupants perceive of motion. It includes five curves: H-90, H-70, H-50, H-30 and H-10, which indicate the percentage of the population who could sense the building motion at the level indicated. For example, H-90 indicated that 90% could perceive motion. The frequency can be measured for each component of horizontal acceleration separately. [26] The peak acceleration is the annual maximum value which makes it possible to compare the values with the ISO 10137 criteria, as shown in the Fig 14.

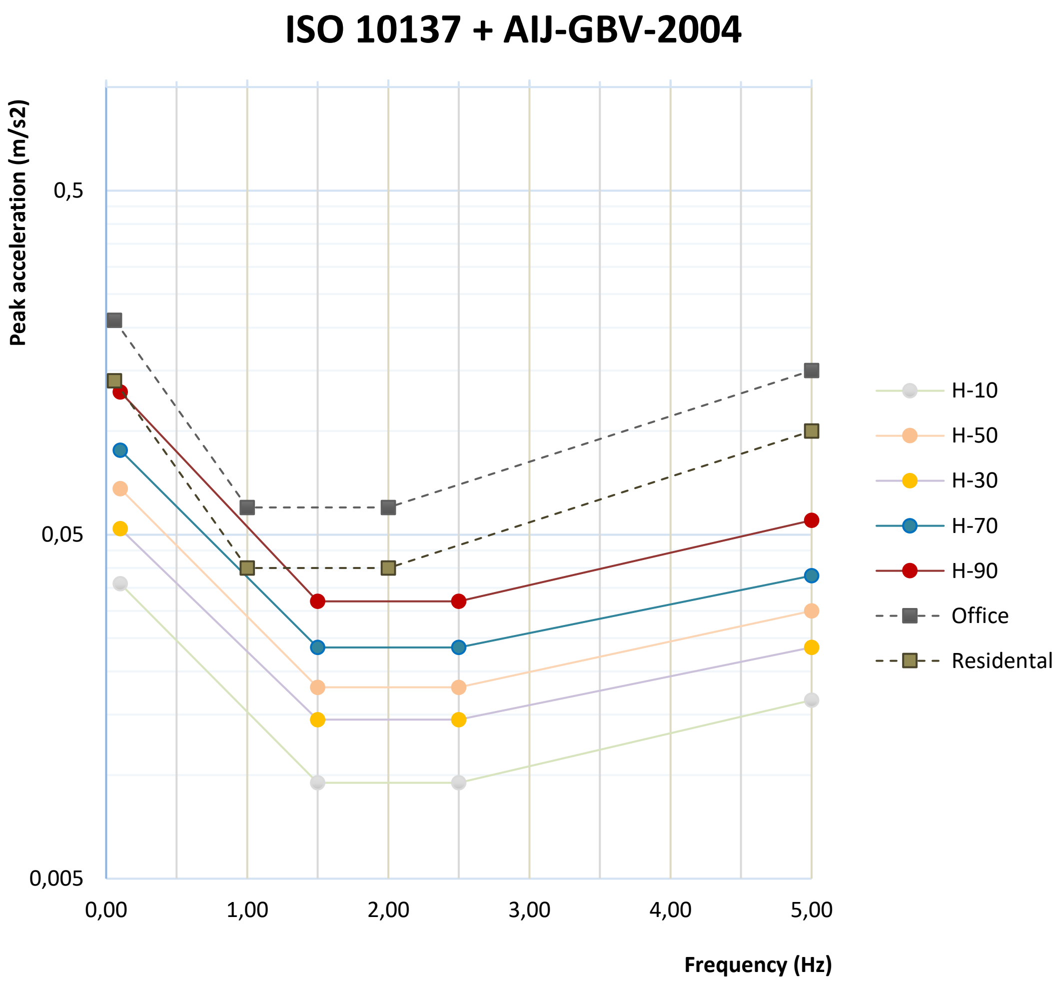

Figure 14. *Combined peak acceleration criteria according to ISO 10137 and AIJ recommendations*

As seen on the comparison, AIJ recommends lower frequencies than the ISO 10137. However, the essential concept of AIJ-GBV-2004 is that the building criteria should be decided by the building owner, since it is difficult to judge the most appropriate vibration level that would fit all building types. [16]

6. COMPARISON AND CALCULATIONS

6.1 CAARC Standard Tall Building

6.1.1 Background and basic parameters

Kwon et. al introduced a comparative example calculation in their study [13] of a Commonwealth Aeronautical Advisory Research Council (CAARC) standard tall building. The CAARC building is previously compared with the AS/NZS 1170.2 and ASCE 7-05 methods to calculate along-wind and cross-wind accelerations, that are provided in the chapter 3. It is also previously calculated using the NALD database method introduced in the chapter 4.1. As a comparison, in this study the same structure is evaluated with the current calculation method of SFS-EN 1991-1-4 for the along-wind direction, to estimate how the results it provides compare with the other standards and the more advanced method of NALD. The same structure is evaluated in two different cases, where the second one represents the same building in 90-degree angle to the first one, which will provide changes in the elastic behaviour of the structure. This is shown in the Fig. 15.

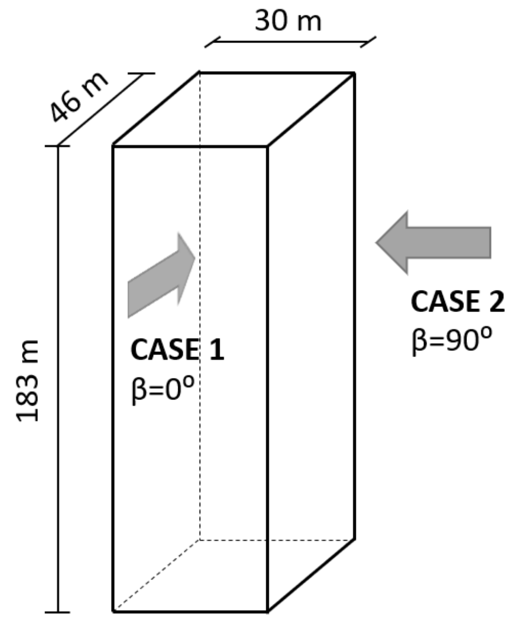

Figure 15. Geometry of the CAARC building and two cases for calculation according to wind direction

As shown in the Fig. 6, the height of the building is 183 m and the cross section is rectangular with the width and depth of 46 m and 30 m. The fundamental frequency of the structure in both directions is $f_x = f_y = 0.2$ Hz and the density of the structure is $\rho_B = 160$ kg/m^3. The force coefficient is $C_D = 1.3$ for the case 1 and $C_{D2} = 1.19$ for the case 2. The mode shapes for all directions are assumed to be linear. The location of the building has been assumed to be Brisbane in Australia and it's assumed to fit the terrain category III according to SFS-EN 1991-1-4. [13]

6.1.2 Calculations of the along-wind accelerations

The calculation procedure is started by determining the mean design wind speed for 1-year return period so that the results can be compared with the ISO 10137 and the AIJ-GBV-2004 comfort guidelines.

Since the SFS-EN 1991-1-4 provides wind speeds considering the climate in Finland, are the calculations also carried out by using the wind speeds of Brisbane presented in the Australian/New Zealand Standard. In Finland, the fundamental value of basic wind speed is $21\ m/s$ which is for 50 years return period. In this study, the calculations carried out using this value are referred as the case A. According to AS/NZS the fundamental value of basic wind speed in Brisbane is $28\ m/s$ for 5 years return period, and this is referred as the case B.

To modify the results for the return period of 1-year, is an annual exceedance probability factor c_{prob} used. The method is carried out by using the recommendation by VTT Technical Research Centre of Finland Ltd [30], which starts from determining the modified basic wind velocity v_b. The results presented here are calculated using the basic wind speed of Finland.

$$v_b = c_{dir} \cdot c_{season} \cdot c_{prob} \cdot v_{b.0} = 15.74\ m/s \tag{5.1}$$

where c_{dir} is the directional factor recommended as 1.0

c_{season} is the seasonal factor recommended as 1.0

c_{prob} is the probability factor defined by (5.2)

$v_{b.0}$ is the fundamental value of basic wind velocity of $21\ m/s$

$$c_{prob} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0.98))}\right)^n = 0.75 \tag{5.1}$$

where K is a shape parameter recommended as 0.2

n is the exponent recommended as 0.5

p is the probability for annual exceeding, recommended as 0.75 for 1-year return period

Now the design mean wind velocity v_m can be calculated using the terrain characteristics for the category III.

$$v_m = c_r \cdot c_0 \cdot v_b = 20.0\ m/s \tag{5.2}$$

where c_r is the directional factor recommended as 1.0

c_0 is the seasonal factor recommended as 1.0

SFS-EN 1991-1-4 uses the reference height of $0.6 \cdot H$ in its calculation procedure, where H is the total height of the structure.

$$c_r = k_r \cdot \ln\left(\frac{z_s}{z_0}\right) = 1.27 \tag{5.3}$$

where k_r is the directional factor recommended as 1.0

c_{season} is the seasonal factor recommended as 1.0

$$k_r = 0.19 \cdot \ln\left(\frac{z_s}{z_{0.II}}\right)^{0.07} = 0.215 \tag{5.4}$$

where $z_{0.II}$ is the roughness length in terrain category II as $0.05\ m$

After determining the design wind speed, the acceleration process is carried out as shown in the chapter 2.5.2. Detailed calculations can be found from Appendix A.

The structural damping is treated differently in the SFS-EN 1991-1-4 so the given critical damping ratio of 0,01 for the structure had to be modified to the logarithmic decrement of structural damping δ_s, that the European Standard uses. SFS-EN 1991-1-4 offers a table with approximate values for different structures, but it can also be calculated using the following equation:

$$\delta_s = \frac{2\pi\eta}{\sqrt{(1-\eta^2)}} = 0.063 \tag{5.5}$$

where η is the critical damping ratio given as 0.01

The logarithmic decrement of damping can now be calculated using the equation given in (2.7):

$$\delta_a = \frac{c_f \cdot \rho \cdot b \cdot v_m(z_s)}{2 \cdot n_1 \cdot m_e} = 0.017$$

Finally, the standard deviation of the along-wind acceleration at the reference height can be calculated using the equation (2.14):

$$\sigma_{a,x}(z) = \frac{c_f \cdot \rho \cdot b \cdot l_v(z_s) \cdot {v_m}^2(z_s)}{m_{1,x}} \cdot R \cdot K_x \Phi_{1,x}(z) = 0.018\, m/s^2$$

To get the peak acceleration $a_{x.max}$, the standard deviation has to be multiplied by the peak factor k_p which is calculated according to the equation (2.11):

$$k_p = \sqrt{2 \cdot \ln(vT)} + \frac{0.6}{\sqrt{2 \cdot \ln(vT)}} = 3.29$$

where T is the average time for the mean wind velocity of 600 seconds according to SFS-EN 1991-1-4

The peak acceleration therefore is:

$$a_{x.max}(z) = \sigma_{a,x}(z) \cdot k_p = 0.058\, m/s^2$$

Using the same procedure for the case B, the peak acceleration was calculated as $a_{x.max2} = 0.083\, m/s^2$. Then the same calculations were repeated for the case 2, which was for the second direction of the wind. All the results are discussed and compared in the next chapter.

6.1.3 Combined results and comparison of the CAARC building

All the results are collected in the Table 4. ASCE and Eurocode only treat the along-wind direction, which is why the cross-wind accelerations are left empty. Also, when comparing the results, some differences between the calculating standards need to be considered. The terrain roughness category used for AS/NZS was terrain category 3 which corresponds to Exposure B in ASCE and terrain category III in Eurocode. The NALD only treats Exposure A and C according to ASCE, which is why the results are the average of these two.

The peak factor is also determined differently. AS/NZS and Eurocode calculate the peak factor for 10-minute period (600 seconds), whereas the NALD and ASCE consider it for 1-hour period (3600 seconds). Also, the accelerations are calculated for different return period: the AS/NZS calculates accelerations for 5-year return period, ASCE and NALD for 10-year return period, and SFS-EN 1991-1-4 for 50-year return period for serviceability design. In the calculations carried out in this study, both wind speeds for of the SFS-EN 1991-1-4 are modified to 1-year return period acceleration as shown in the previous chapter 5.3.2, so that the highest results can be compared with the ISO 10137 and AIJ-GBV-2004 guidelines in the Fig. 16. All in all, due to these differences the results cannot be directly compared with the previous calculations carried out using the AS/NZS, ASCE and NALD.

The results calculated in this study are referred as SFS-EN for both cases A and B and cases 1 and 2 as seen on the Table 4. The results of the case B, where the actual wind speed of Brisbane was used, are quite close to the previous calculations. The peak acceleration for the case A, which used the wind speed of Finland, are notably lower, which shows that the basic wind speed used in calculations has a large impact on the final results. According to the results, it could be concluded that considering wind-induced vibration, a high-rise building as the CAARC could be built in Finland since all the results of case A fit the ISO 10137 and AIJ-GBV-2004 criteria well. The case B however requires further investigation, since the peak vibrations are higher.

Table 4. *Peak acceleration result comparison of the CAARC Building*

CASE 1	*Responses*	***AS/NZS***	***ASCE***	***NALD***	***SFS-EN (A)***	***SFS-EN (B)***
	Along-wind peak accel. [m/s^2]	0,081	0,068	0,084	0,058	0,083
	Cross-wind peak accel. [m/s^2]	0,152	-	0,113	-	-
CASE 2						
	Along-wind peak accel. [m/s^2]	0,054	0,046	0,058	0,036	0,052
	Cross-wind peak accel. [m/s^2]	0,171	-	0,116	-	-

One interesting thing to note considering the previous results of the CAARC building, is that the cross-wind peak accelerations are considerably higher than the along-wind accelerations. The previous cross-wind accelerations calculated by AS/NZS differ quite remarkably from the values of the NALD database, while the along-wind acceleration values are very close. One reason for this difference could be the difficulty to bring the cross-wind vibration phenomenon to a standardized level, which is why the AS/NZS provides very conservative results, whereas the NALD uses data from previous wind tunnel tests.

For further comparison, the results are calculated using different terrain category. Sometimes the correct terrain might be hard to evaluate, and the criteria given by design standards is not always clear. However, as seen on the Table 5, using different terrain category has a great impact on the final results. The results are from the case B using the slenderness factor of 2.0.

Table 5. *Comparison using different terrain category according to SFS-EN 1991-1-4*

CASE 1	*Responses*	***Terrain II***	***Terrain III***	***Terrain IV***
	SFS-EN (B), Along-wind peak accel. [m/s^2]	0,094	0,083	0,069
CASE 2				
	SFS-EN (B), Along-wind peak accel. [m/s^2]	0,059	0,052	0,043

For final comparison, the results carried out by the SFS-EN 1991-1-4 are now carried out using different slenderness factor. The results in Table 4. are for the slenderness factor ζ of 2.0, which is the recommended value for towers and chimneys, which could also resemble a slender high-rise building. However, since there are no clear criteria for selecting the right value for the factor, as a comparison, the calculations for the case B are also carried out using different values for it as seen on the Table 6, which showcases its impact on the final results.

Table 6. Comparison using different slenderness factors according to SFS-EN 1991-1-4

CASE 1	*Responses*	***ζ = 1.0***	***ζ = 1.5***	***ζ = 2.0***	***ζ = 2.5***
	SFS-EN (B), Along-wind peak accel. [m/s^2]	0,072	0,078	0,083	0,086
CASE 2					
	SFS-EN (B), Along-wind peak accel. [m/s^2]	0,045	0,049	0,052	0,054

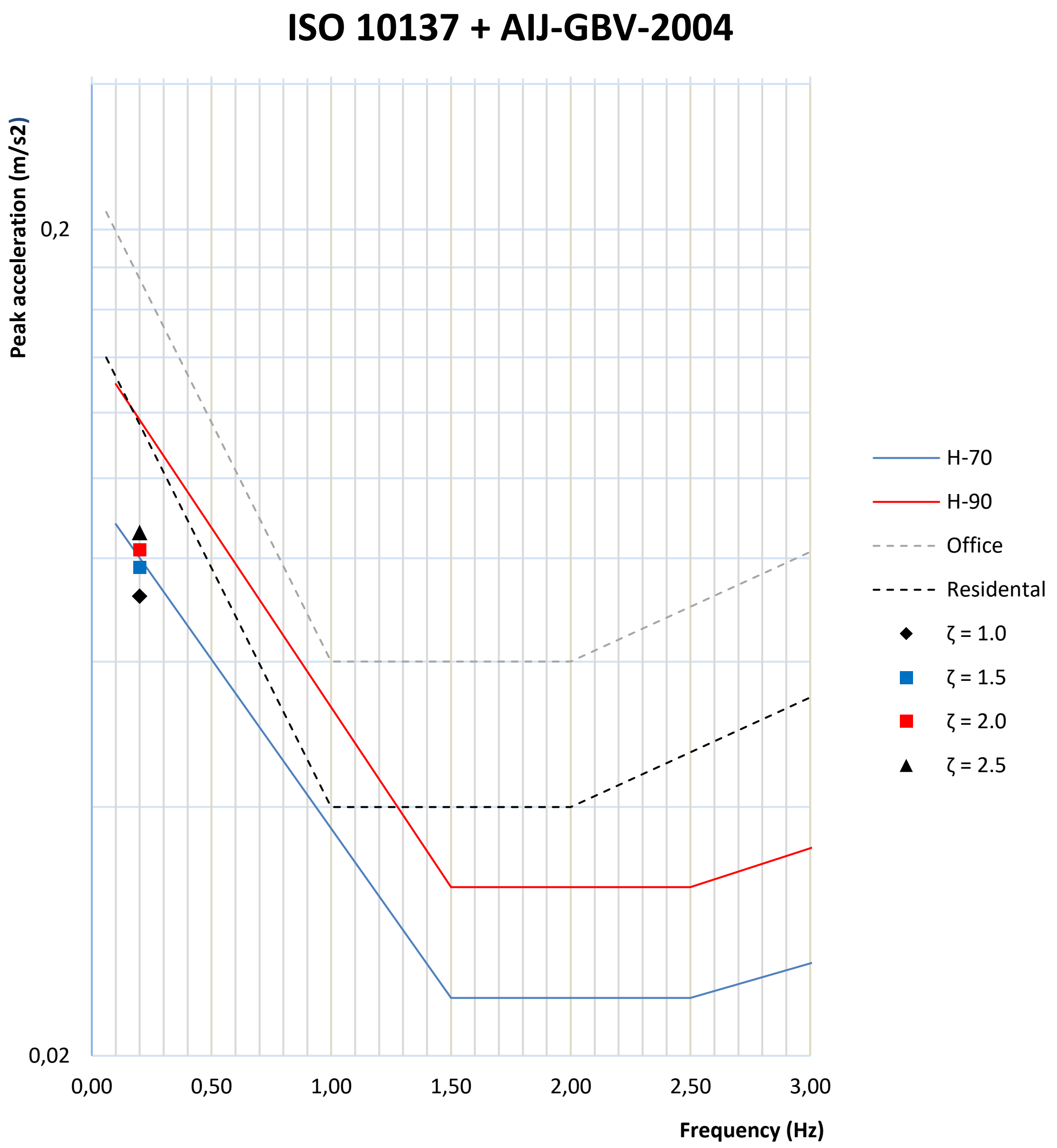

Figure 16. *Results calculated by SFS-EN 1991-1-4 for Case B using different slenderness factors, compared with the ISO 10137 and AIJ-GBV-2004 guidelines*

According to the Fig.8, all values of case B fill the comfort requirements of ISO 10137 and are close to the H-70 line of AIJ-GBV-2004, which means that roughly 70% of the occupants might sense some vibrations in case of the wind conditions that determined the peak accelerations. Choosing the right value for the slenderness factor can also be considered to be quite important for the final results but does not have as big impact as using the correct design wind speed and selecting the correct terrain category.

6.2 Tall building according to NALD Experimental Data

6.2.1 Background and basic parameters

Another example building is provided by the NALD database. The first version of this example building was calculated in 2003 [22] and later the results were revisited during the update of the database. The first version provided spectral amplitude at a specified reduced frequency, which required the user to manually calculate the accelerations and moments, but in the later version all the results are computed by the updated analysis module of the database. [13] The motivation of this calculation is to compare the AIJ Recommendations results with the NALD database. This building has calculated data in all three response directions, which is why the AIJ Recommendations' method is the best suitable standardized option. The main focus in the calculations is to use similar values for design basis as in the previous results.

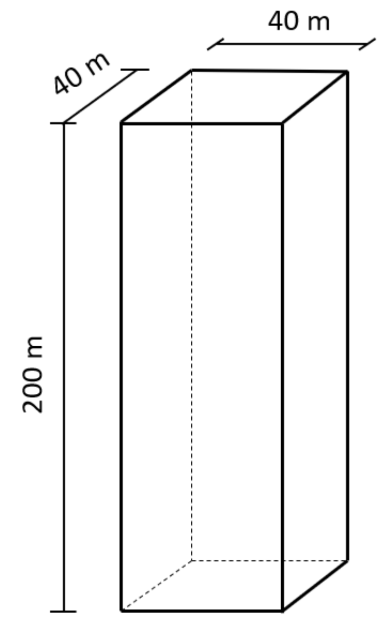

Figure 17. *Geometry of the NALD example building*

The NALD example building is 200 m in height and it has a square cross section with a width of 40 m. The natural frequencies of the building in the along-wind, cross-wind and torsional directions are previously found to be $f_x = f_y = 0.2\ Hz$ and $f_z = 0.35\ Hz$. The density of the building is $\rho = 250\ kg/m^3$. The critical damping ratio is $\xi = 0.02$ and drag force coefficient is $C_D = 1.3$. The structure is located in an urban area, which translates to terrain category IV according to AIJ.

The results compared in this study are the RMS moment coefficients and the peak accelerations on the top of the building. The NALD uses 10-year return period for serviceability designs, which was used to calculate the previous results. The AIJ Recommendations uses 100-year return period, but the 10-year return period wind speed provided by NALD in the previous calculations, which was $U_{3s} = 37.96\ m/s$. This value is used in the calculations for better comparison. However, since the wind speed provided is the 3-s gust wind speed, it needs to be modified to the 10-min mean wind velocity used in the AIJ calculation system. For the modification, a method provided by Durst [30] is used, that is based on the statistical analysis of meteorological wind velocity records. This method uses a conversion factor $G_{v(10-\mathrm{min})}$ that converts the 3-s gust speeds into the 10-minute mean wind velocity.

$$U_{10min} = \frac{U_{3s}}{G_{v(10-\mathrm{min})}} = 26.76\ m/s \tag{5.6}$$

where $G_{v(10-\mathrm{min})}$ is the conversion factor of 1.42

6.2.2 Calculating the wind-induced vibration

First the RMS moment coefficients are calculated for each direction of vibration. They are determined by the terrain characteristics and the geometry of the building. The detailed equations are explained in the chapter 3 and calculations are found from the Appendix B. RMS moment coefficient for the NALD example building in the along-wind direction is:

$$C'_g = 2I_H \frac{0.49 - 0.14\alpha}{\left\{1 + \frac{0.63\left(\sqrt{BH}/L_H\right)^{0.56}}{(H/B)^k}\right\}} = 0.095$$

where I_H is the turbulence intensity at reference height of 0.138

α is the exponent of power law for wind profile as 0.27

k is a structural factor of 0.07 if $H/B \geq 1$
L_H is the turbulence scale at reference height as $258\ m$

The cross-wind and torsional RSM overturning moment coefficients are only determined from the geometry of the structure. First the cross-wind coefficient is calculated as:

$$C'_L = 0.0082(D/B)^3 - 0.071(D/B)^3 + 0.22(D/B) = 0.157$$

where D is the depth of the building of $40\ m$
B is the width of the building $40\ m$

The torsional RMS moment coefficient is:

$$C'_T = 0.04\left(\frac{D}{B}\right)^2 + 0.02 = 0.06$$

where D is the depth of the building of $40\ m$
B is the width of the building $40\ m$

Next, the design process for peak accelerations stars by determining the design wind speed $U_{H.s}$ and then the design velocity pressure of the wind q_H. The basic wind speed for 10-minute mean wind is provided in the previous chapter and this is then modified to the value used in calculations using the parameters by AIJ Recommendation. Since the designing code itself focuses on wind speeds and results used in Japan, a recommended equation by Kwon and Kareem [31] is used to calculate the design wind speed for serviceability design that can be compared with the previous results by NALD:

$$U_{H.s} = \beta \cdot \left(\frac{Z}{10}\right)^{\alpha} \cdot U_{10min} = 34.61\ m/s \tag{5.7}$$

where β is the exponent of power law, recommended value 0.576 for terrain category IV
Z is the height of observation, which is the height of the building as $200\ m$
α is the exponent of power law, recommended value $0{,}27$
U_{10min} is the modified 10-minute mean wind velocity of $26.76\ m/s$

$$q_H = 0.5 \cdot \rho \cdot (U_{H.s})^2 = 0.731\ kN/m^2 \tag{5.8}$$

where ρ is the air density as $1.22\ kg/m^3$
$U_{H.s}$ is the design wind speed as $34.61\ m/s$

After having the design wind speed and pressure, the along-wind calculation process is carried out according to the method explained in the chapter 3. The averaging time of the acceleration is calculated for both 1-hour (3600 seconds) and 10 minutes (600 seconds). The 1-hour averaging time is used in the NALD results and 10 minutes is usually used in the AIJ system. The calculation results for the 1-hour averaging time are provided here. The peak factor g_{aD} and peak acceleration in the along-wind direction a_{Dmax} is defined by the following equations:

$$g_{aD} = \sqrt{2\ln(Tf_D) + 1.2} = 3.789$$

where f_D is the natural frequency of the first mode in along-wind direction as $0.2\ Hz$
T is the averaging time of evaluation as $3600\ s$

$$a_{Dmax} = \frac{q_H g_{aD} B H C_H C'_g \lambda \sqrt{R_D}}{M_D} = 0.035\ m/s^2$$

where g_{aD} is the peak factor for along-wind vibration as 3.789
q_H is the design velocity pressure as $0.731\ kN/m^2$
B is the building width as $40\ m$
H is the mean roof height of the building as $200\ m$
C_H is the wind force coefficient C_D at reference height as 1.3
C'_g is the rms overturning moment coefficient as 0.095
λ is the mode correction factor as 1.221
R_D is the resonance factor as 0.699
M_D is the generalized mass of the building as $80\ 000\ 000\ kg$

Next, the cross-wind peak factor g_L and peak acceleration $a_{L.max}$ are calculated. The calculation method follows the one provided in the chapter 3. It can be noticed that for cross-wind direction, more factors, such as the non-dimensional exponents of power law β_1 and β_2 are determined by numerical calculations.

$$g_L = \sqrt{2\ln(Tf_L) + 1.2} = 3.827$$

where f_L is the first mode natural frequency of vibration of a structure in the crosswind direction as $0.2\ Hz$
T is the averaging time of evaluation as $3600\ s$

$$a_{L.max} = \frac{q_H g_L B H C'_L \lambda \sqrt{R_L}}{M_L} = 0.089\ m/s^2$$

where q_H is the design velocity pressure as $0.731\ kN/m^2$
g_L is the peak factor for the cross-wind vibration as 3.827
B is the width of the building as $40\ m$
H is the mean roof height of the building as $200\ m$
C'_L is the rms overturning moment coefficient as 0.157
λ is the mode correction factor 1.221
R_L is the resonance factor as 2.759
M_L is the generalized mass of the building as $80\ 000\ 000\ kg$

Finally, the torsional direction is evaluated by calculating the torsional peak factor g_T and peak angular acceleration $a_{T.max}$. In the torsional directions, the calculations require more values that are selected from tables in the AIJ Recommendations compared to the along-wind and cross-wind directions. The peak acceleration is given in angular acceleration that can be then divided into the along-wind and cross-wind components. The natural frequency of the building in the torsional direction was slightly higher than for the along-wind and cross-wind directions.

$$g_T = \sqrt{2\ln\left(Tf_T\right) + 1.2} = 3.934$$

where f_T is the first mode natural frequency of vibration of a structure in torsional direction as $0.35\ Hz$
T is the averaging time of evaluation as $3600\ s$

$$a_{T.max} = \frac{0.6 q_H g_T B^2 H C'_T \lambda \sqrt{R_T}}{I_T} = 0.001151\ rad/s^2$$

where q_H is the design velocity pressure as $0.731\ kN/m^2$
g_T is the peak factor for the torsional vibration as 3.934
B is the width of the building $40\ m$
H is the mean roof height of the building $200\ m$

C'_T is the rms overturning moment coefficient as 0.06
λ is the mode correction factor as 1.221
R_T is the resonance factor as 0.369
I_T is the generalized mass of the building for torsional vibration as $2.133 \cdot 10^{10}\ kgm^2$

6.2.3 Combined results and comparison of the NALD example building

The results calculated using the AIJ Recommendations for Loads are now compared with the existing data provided by the NALD database. The results are calculated for the wind speeds of 10-year return period, which is why they cannot be directly compared with the ISO 10137 or AIJ-GBV-2004 comfort guidelines. However, if the results were to modify to the 1-year return period, the peak accelerations would be lower due to the decrease in the design wind speed. It could be assumed, that this specific building would stay withing the comfort guidelines in all directions of vibration.

The combined results are provided in the Table 7. The results calculated using the AIJ Recommendations are provided for both 1-hour and 10-min averaging times. The former is compared with the existing results from NALD database.

Table 7. *Peak acceleration and RMS moment coefficient result comparison for the NALD example building*

		NALD	**AIJ**
Along-wind			
	RMS moment coefficient $C'_{M.g}$	0,109	0,095
	Peak acceleration (1-h averaging time) [m/s^2]	0,0368	0,035
	Peak acceleration (10-min averaging time) [m/s^2]	-	0,030
Cross-wind			
	RMS moment coefficient $C'_{M.L}$	0,133	0,157
	Peak acceleration (1-h averaging time) [m/s^2]	0,061	0,089
	Peak acceleration (10-min averaging time) [m/s^2]	-	0,77
Torsional			
	RMS moment coefficient $C'_{M.T}$	0,044	0,06
	Peak angular acceleration (1-h averaging time) [rad/s^2]	0,00121	0,00115
	Alongwind component [m/s^2]	0,024	0,0228
	Cross-wind component [m/s^2]	0,024	0,0228
	Peak angular accel. (10-min averaging time) [rad/s^2]	-	0,00101

It can be noticed that the differences in the values for the cross-wind and torsional directions are higher than for the along-wind direction. This is true for both the RMS moment coefficients and the peak acceleration. Generally, the results in the along-wind direction has been proven to be more congruous with the collected and calculated data as discussed in the chapter 3 and 4 and the design standards also seem to give more coincident results in this direction. Along-wind direction is the most researched and well covered direction in most designing standards, which is why it is natural that the methods are more advanced than in the other vibration directions. Also, the effects causing along-wind vibration are easier to estimate, compared to the cross-wind and torsional directions.

However, as seen from the results, the along-wind peak acceleration is slightly lower than the value according to the NALD database, whereas the cross-wind and torsional directions provide higher values. The calculation methods for both of these directions used some simplified factors that might lead to more conservative results. In general, it is better for a design standard to provide too conservative results rather than results that are lower than in the reality.

It can also be noted that the averaging time clearly affected the final results. The averaging time on 10-minutes provides lower accelerations than the 1-hour alternatives, which is due to the decrease in the peak factors.

Finally, as well as for the CAARC building, also the NALD example building has the highest wind-induced acceleration in the cross-wind direction. This shows, that estimating the peak accelerations especially in this direction is important for high-rise building design. Compared with the CAARC building, the accelerations in the second example are significantly lower. One factor that has a big impact on this is the density of the building, which was much higher in the NALD with the value of $250\ kg/m^3$ compared with the CAARC building of $180\ kg/m^3$. This affects to the total mass of the building, which is used to estimate the peak accelerations. The total mass of the building is not described in the design standards and it is not clear, if the live loads are considered while estimating the accelerations.

In the AIJ Recommendations, the exponent of power law for first mode of vibrations, referred as β was also slightly unclear. In the commentary part of AIJ Recommendations for Loads [16] it is stated that according to wind tunnel test results of a rectangular section of $D/B = 0.2 - 0.5$ the exponent had found to be $\beta = 0.2 - 4$ [16]. In this example the cross section was a square. Eventually, the value provided by Kwon's and Kareem's [31] comparison was used. The factor affected significantly to the results of the peak accelerations, which is why it should be treated carefully in the calculation process.

7. CONCLUSIONS

This study shows that the wind-induced vibration is a challenging phenomenon, that requires vast knowledge of structural behaviour and usage of different calculation parameters. It occurs mainly on slender structures, such as high-rise buildings, that have low fundamental frequencies and are subjected to various wind phenomena such as vortex effects in the higher levels of the structure.

Wind-induced vibration occur in three directions, which are the along-wind, cross-wind and torsional components. If the building fits certain criteria according to the relation of height, shape and area of the cross-section, or has high eccentricity between the elastic and mass centres, it is necessary to determine the wind-induced vibration in all its three components, as cross-wind or torsional directions could be the prior cause of fluctuation in the structure.

Current criteria and requirements for in depth dynamic response analysis vary in different publications, but a clear limit for human perception of building motion is found to be frequencies up to 1 Hz. This can be used as a guideline for carrying out a vibration analysis, but cases should always be evaluated independently, using criteria that best fit the characteristic of the structure and its location.

Selecting correct calculation parameters when using standardized methods and knowing the real wind conditions in the region of the designed structure are important for getting accurate results. The standardized methods regarding cross-wind and torsional evaluation still rely greatly on simplifications, and the results might not always reflect the reality of the situation. For this reason, carrying out comparison calculations using database-enabled design frameworks, such as the NALD, might be useful and provide more realistic values. The most recommended practice for advanced structures is still to use wind tunnel testing for determining the most accurate dynamic reactions, but also their usage require experience and knowledge for understanding the results correctly.

One of the most advanced structural design standard in terms of wind-induced vibration at the moment is the 2015 version of AIJ Recommendations for Loads, since it offers a calculation method for all three direction of the wind-induced acceleration. However, as seen on the example calculations and other comparisons in the past, it does not always provide very accurate results in the more complex directions of cross-wind and torsional accelerations, that are more dominant for the wake-induced effects.

The example cases also show, that when using different standards for calculations, some modifications for the calculation parameters are required. This is especially true for determining the correct design wind speed. In the along-wind direction the results conducted from different designing standards seem to be the most coincident.

It could be assumed, that the cross-wind and torsional directions are slowly added to new releases of the main structural design standards around the world, due to the excessive research of their importance in high-rise building design. This would provide more research and comparison material in the future, in terms of finding more accurate methods for the cross-wind and torsional acceleration evaluation. Also, the combination of these three components and its relation to the wind pressures are still areas that require further research. Bringing this phenomenon into a FEM-designing state could also provide valuable comparison material for the current standardized calculation methods.

Carrying out more comparison calculations for different heights and shapes of buildings using multiple standards could also bring better understanding of the differences in the standards. Comparing these results with the existing criteria and guidelines provided in the standards could offer valuable information for the early stages of a high-rise building designing process.

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